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EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH.

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WITH DISCUSSION.

Through the courtesy of C. K. Bannister, M. Am. Soc. C. E., and at his suggestion, the authors were enabled to carry out, in August, 1897, a limited series of experiments on the flow of water in the recently finished conduit of the Pioneer Electric Power Company, of Ogden, Utah, of which he was at that time chief engineer. As the main dam has not been constructed, it was not possible to extend the experiments to the velocities of flow corresponding to the full carrying capacity of the conduit. It is the intention to do so when opportunity offers. Believing that the results obtained so far, and a description of the method of experimentation adopted, may prove of interest to the profession, this paper is submitted.

For a description of the entire pipe line and of the construction of the pipe, reference may be made to a paper recently published.* The plan and profile of each of the portions experimented upon are given herewith, in connection with the account of the experimental results.

I.—Description of Object and Methods.

General Plan.

The main object of the experiments was to determine the relation between the rate of discharge of the pipe and the loss of head between certain definite points. To accomplish this it was necessary to measure, simultaneously, the pressure at each end of the length of pipe under experiment, and the rate of discharge.

As a secondary object, observations were taken on the loss of head in the Venturi meters, with different velocities of flow.

Pressure Measurements.

(*1*) *Method Adopted.*—A consideration of the probable range of the losses of head to be measured made it evident that a far greater degree of precision was required in the pressure measurements than could be attained with any pressure gauge of the Bourdon type. It seemed reasonably certain that the best results attainable with such a gauge would be unreliable by at least as much as 1 lb. per square inch (equivalent to a water column of 2.3 ft.); more probably the errors would be two or three times that amount, or even greater. Such a degree of uncertainty would render the results wholly valueless. It was therefore decided to use mercury gauges.†

(*2*) *Description of Gauges.*—The form of gauge used was the ordinary open manometer. Fig. 1*a* shows the essential parts of the manometer as used at the point of greatest pressure, near the power house. Besides the manometer proper, the figure shows a water-jacket surrounding the mercury tube, and a vessel into which the water passing

* "The Power Plant, Pipe Line and Dam of the Pioneer Electric Power Company at Ogden, Utah," by Henry Goldmark, M. Am. Soc. C. E. *Transactions*, Vol. xxxviii, p. 346.

† This discussion of Bourdon gauges has reference only to their applicability to this series of experiments. It is not meant to imply that such gauges can never give satisfactory results in experiments of this kind. For a great length of pipe the loss of head may be so great that the errors of a Bourdon gauge are relatively unimportant. In any case, however, such gauges should be repeatedly and carefully tested, during the period of experimenting, at about the pressures at which they are used. It will aid in eliminating gauge errors if the pressure is measured at various points along the profile of the pipe.

through the water-jacket wasted; the mercury reservoir, being placed in this vessel and being surrounded by water. The mercury reservoir A is of cast iron, the interior being cylindrical, 4 ins. in diameter. In one side is a window, fitted with glass, through which the position of the mercury surface may be observed. The vertical tube $B\,B$ is of ¼-in. wrought-iron pipe, except at the upper end, where a length of glass tube D is attached, with a scale for reading the position of the top of the mercury column. Into the top of the reservoir A is fitted a pipe $C\,C$, leading to that cross-section of the main pipe at which the pressure is to be measured. In the pipe $C\,C$ is a gate-valve Y, and at

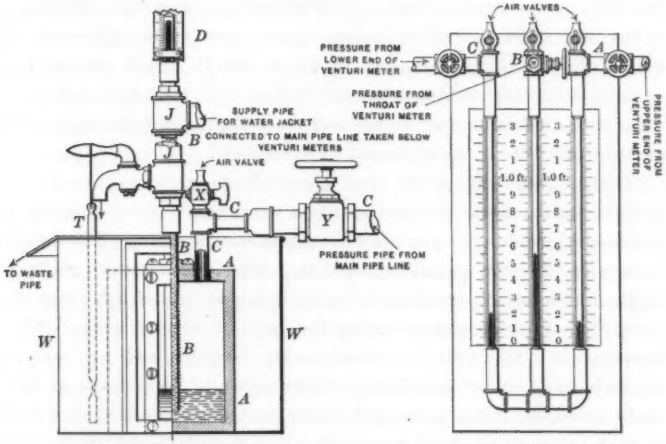

FIG. 1a. FIG. 1b.

X is an air-cock. When the gauge is in use the pipe $C\,C$ and the space in the reservoir above the mercury are filled with water. The air in the reservoir is removed by opening the air-cock X and allowing water to enter slowly through the partially open valve Y. The air-cock is then closed and the valve Y fully opened, and mercury rises in the tube $B\,B$. The height of the mercury column measures the water pressure at the surface of the mercury in the reservoir; and thus, by allowing for difference of level, the pressure is known at any point of the pipe $C\,C$ and of the cross-section of the main pipe to which this pipe is attached.

Two questions in connection with the use of the gauge call for some explanation—the effect of changes of temperature upon the mercury column, and the method of attachment of the gauge to the main pipe.

(*3*) *Correction for Temperature.*—In order that the observations may be comparable, the observed height of the mercury column must in every case be reduced to that of an equivalent column at some standard temperature. It was therefore necessary to know the temperature of the mercury at the time of each observation of its height. The degree of precision needed in the temperature observations may be estimated by noticing that a change of 1° Cent. affects the length of the mercury column by about $\frac{1}{5550}$ of its value. The static pressures at the four points at which measurements were made (expressed in height of water column) were about 462 ft., 153 ft., 101 ft. and 48 ft., respectively. Temperature changes of about 1.2°, 3.6°, 5.5° and 11.5° Cent. at these four points, respectively, would thus each correspond to an error of 0.1 ft. in the measured pressure-head.

At the lower end of the steel pipe, where the pressure had its greatest value, it was decided to use a water-jacket for the purpose of keeping the mercury at a nearly constant temperature. This jacket is shown in Fig. 1*a*, already referred to. The ¼-in. mercury tube was enclosed in a larger pipe *J J*, through which a stream of water was allowed to flow continuously during the time of experimenting. The temperature of this water remained nearly constant, and was read at intervals by means of a chemical thermometer *T*, suspended in the waste-vessel *W*. This water-jacket surrounded a length of about 30 ft. of the column, the total length of which varied from 34 ft. to 32.8 ft. A length not exceeding 4 ft. was thus left unjacketed; but it is believed that no serious error was thus introduced. Thus, for a length of 4 ft. of mercury, an error of about 10° Cent. would be required to introduce an error of 0.1 ft. (water) into the pressure-head.

At the other pressure stations the heights of the mercury columns were much less, and it was thought sufficient to determine the temperature by means of a thermometer placed beside the manometer tube. During most of the observations taken at the upper end of the steel pipe line, where the greatest height of the mercury was about 11.3 ft., the instrument was shaded during the time of experimenting. At the upper station on the wooden pipe line the manometer was fully

shielded from the sun by a shed built for the protection of the pipe against rock slides, and the temperature of the air was very constant. At the lower station on the wooden pipe line the tube containing the mercury column was shielded from the sun by a 2-in. plank upon which it was mounted, and the temperature was read from a thermometer hanging beside it.

Although, in some respects, more favorable temperature conditions would have been desirable at two of the manometer stations, it is not believed that serious error was introduced into the results from this cause.

(*d*) *Attachment of Manometers to Main Pipe.*—The proper method of attachment of a piezometer for the purpose of measuring the pressure at any section of a pipe carrying water has been the subject of considerable discussion. The main questions raised have been: (*a*) whether a single point of attachment to the pipe is sufficient, or whether attachment should be made to a chamber communicating with the pipe at several points of the circumference; and (*b*) whether in case attachment is made only at a single point, this should be at the top or at some other point on the circumference. A careful consideration of the principles involved led the authors to the conclusion that the number of points of attachment is immaterial, and that the position of the point of attachment on the circumference is important only as affecting the liability to the collection of air in the pipe leading to the piezometer.*

In the case of the steel pipe the simplest method available was to make connection with relief valves at the top of the pipe. At the lower end the manometer was located in the power house, about 200 ft. from the point of attachment to the main pipe; the connecting pipe, $\frac{3}{4}$ in. in diameter, entering the building through a window. At the highest point of this connecting pipe was placed a valve for drawing off air. At the station near the upper end of the steel pipe line the connection with the manometer was made through a pipe about 20 ft. in length, with a valve for air at the highest point. In both cases the air valves were opened at intervals during each run, in order to remove any possible accumulation of air.

In connecting the manometer to the wooden pipe, a hole was bored at right angles to the axis of the pipe. Into this hole the pipe lead-

* A further discussion of these questions is given in the appendix to this paper.

ing to the manometer was screwed, care being taken that there should be no projection beyond the inner surface. At each of the pressure stations on the wooden pipe line the length of the connecting pipe was only a few feet.

On Plate XXXVII, Figs. 1 and 2 show the upper and lower manometers, respectively, on the wooden pipe; and on Plate XXXVIII, Figs. 1 and 2 show the upper and lower manometers, respectively, on the steel pipe. The difference-gauges on the north and south Venturi meters, respectively, are shown by Figs. 1 and 2, Plate XXXIX.

Measurement of Rate of Discharge.

(*1*) *Method Adopted.*—The main pipe, 72 ins. in diameter, divides near the power house into two branches, each 54 ins. in diameter. Each of these branch pipes is furnished with a Venturi meter with automatic register. These meters were the only available means of measuring the rate of discharge with any approach to precision.

The automatic registers, however, were not adapted to the purpose in hand. The register is an integrating device, recording the total quantity discharged, but not showing the rate of discharge directly. The rate of discharge can be determined from its readings only by observing the difference between the readings at the beginning and end of a known period of time. This period must be of considerable length, since the smallest quantity directly indicated by the register is 10 000 cu. ft.; and intermediate readings cannot be estimated with precision. Moreover, the result of such a determination is not reliable unless the rate of discharge has remained uniform, since the indicating mechanism moves only once in ten minutes, registering at each movement an amount proportional to the rate of discharge at that instant. The register itself gives no indication as to what fluctuations in the rate of discharge occur during the ten-minute intervals. It was therefore decided to attach to each meter a mercury gauge, designed to show, at every instant, the difference between the pressures existing at the throat and the up-stream sections of the Venturi. This pressure difference indicates the rate of discharge instantaneously; no time interval need be measured.

(*2*) *Difference-Gauges.*—For accomplishing the object stated, the essential parts of the difference-gauge are two vertical glass tubes

FIG. 2.

FIG. 1.

connected at the bottom, and at the top communicating respectively with the two sections whose pressure difference is to be determined. The tubes are filled with mercury to a convenient height, the space above the mercury and the connecting pipes being filled with water. By means of a fixed vertical scale, the difference in level between the tops of the two mercury columns may be read at any desired instant.

The gauges, as actually constructed, were designed to accomplish an additional object, namely, to indicate the total loss of head in the meters. For this purpose a third tube was added, connected with the other two at the lower end, and at the top communicating with the 54-in. pipe immediately below the meter. Simultaneous readings of the three mercury columns thus served to determine both (*a*) the instantaneous rate of discharge, and (*b*) the corresponding loss of head in the meter.

In Fig. 1*b* are shown the essential features of these difference-gauges as actually constructed and used. The three glass tubes communicate with each other freely at the bottom through suitable connections of wrought-iron pipe. At the top they communicate with separate pipes *A*, *B*, *C*, leading respectively to the three sections whose pressures are to be compared. Each is furnished at the highest point with an escape cock for drawing off air, and in each pipe is placed a gate-valve.

In attaching the gauge for use, the air-valves *V* were at first left open, and the space in the tubes above the mercury completely filled with water, the gate-valves in the connecting pipes being, for this purpose, only slightly opened. These latter valves were tightly closed as soon as all the air had escaped from the tubes and connecting pipes, and very gradually opened again after the air-valves had been closed. No difficulty was experienced from air in the pipes *A*, *B*, *C*. In two of these pipes (*A* and *B* in each gauge) the air-valve was at the highest point; in the third (that running to the section below the meter) this was not the case. Whether air was present in any pipe could be tested at times of no discharge, since under static conditions the three mercury columns must stand at the same level if the connecting pipes were completely filled with water. During the series of experiments there were frequent opportunities of applying this test, and air was never found to be present, even in the pipe with the upward bend,

except in two or three instances. These instances occurred after the adjacent receiving chamber had for some reason been emptied of water; and it was always found easy to blow off the air through the air-valve *V*.

The scales for reading the heights of the mercury in the tubes were graduated to hundredths of a foot, and the third decimal place was estimated in taking readings. In setting up the instrument, the adjustment was determined by the equality of the readings of the three scales under static conditions.

Programme of Tests.

In carrying out the experiments, an observer stationed at each of the gauges in use took readings at short intervals during a certain time previously agreed on. The interval between readings was originally intended to be one minute, but it was found that under steady conditions it was not always necessary to read so frequently.

The rate of discharge was controlled by the superintendent in charge of the power house. The aim was to maintain a uniform flow for a certain period; then to change the rate of discharge and maintain a uniform flow for another period; then change again; and so on as long as desired. In this way was obtained a series of simultaneous sets of values of the three quantities sought, the rate of discharge and the pressures at two sections of the main pipe.

It was not necessary to note the time of reading with great precision, so long as nearly uniform conditions of discharge and pressure could be maintained. A sufficient degree of uniformity was secured without difficulty, except in two or three instances to be mentioned later.

After a change in the rate of discharge, some time always elapsed before a steady condition was re-established. It was found that about fifteen minutes must be allowed for each change. The first day's results were unsatisfactory because sufficient time was not allowed for steady conditions to be re-established after changes in the rate of discharge. During the period of adjustment from one steady condition to another, the pressure gauges showed marked fluctuations, the range being at first considerable,* but decreasing and nearly disappearing

*Although no accurate determination was made of the increase of pressure caused by sudden stoppage or change of the rate of discharge, it is certain that, even at the lower end of the line, the mercury column in no case rose more than 3 ft. above the static height. In one case a very sudden stoppage occurred, owing to the blowing out of a fuse, but the above statement holds even for this extreme case.

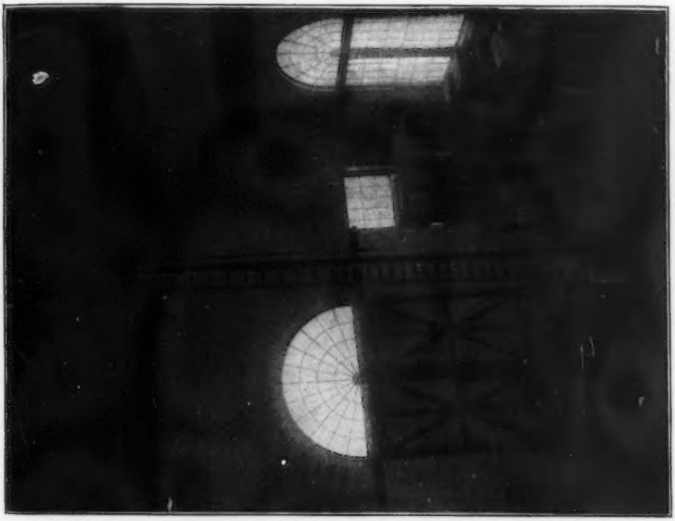

FIG. 2.

FIG. 1.

after ten or fifteen minutes. The difference-gauges attached to the Venturi meters, on the other hand, adjusted themselves to changed conditions much more quickly, their oscillations at times of change continuing not more than two or three minutes. This showed that the rate of discharge became steady very quickly, while the pressure continued to oscillate for some time. It was at first suggested that the observed oscillations of the manometer columns might be due to the inertia of the mercury. It was found, however, by plotting curves showing the simultaneous readings of the two gauges in use at the same time, that there was always a close agreement between the periods of oscillation of the two columns. This is shown in Fig. 5. It would therefore appear that the observed oscillations correspond to true oscillations of pressure in the pipe. Doubtless these variations of pressure are transmitted in waves throughout the length of the pipe. No difference of phase between the oscillations of pressure at the two stations could be detected.*

Because of conditions existing at the inlet, the highest rates of discharge observed were accompanied by a decrease of pressure throughout the pipe. Under such circumstances it was of more importance to secure exact coincidence in the times of observing the two pressures, since any uncertainty as to what observations at the two stations are simultaneous introduces an uncertainty into the value of the loss of head between the stations. It was found that the gauges at the two stations always gave practically identical determinations of the rate of decrease of the pressure. This is illustrated by the plotted results shown in Fig. 4.

It will be noticed that the oscillations of pressure at the beginning of the period represented by the curves in Fig. 5 show a period between successive maxima or minima of about six minutes. The records for the entire series of experiments show a period lying between five and six minutes. It should be said that there were also minor oscillations of a period so short that no attempt was made to record them. The period of these minor oscillations was not determined, but it was only a few seconds.

In cases of falling pressure the oscillations recorded at the two stations serve as a check upon the coincidence of the instants of observation.

* The velocity of propagation of a pressure wave should, in fact, be practically equal to the velocity of sound in water—about 4 700 ft. per second. The interval occupied in traversing the distance between the two manometers would therefore be less than one second, since the greatest length of pipe experimented upon was 4 427 ft.

Reduction of Observations.

(1) *Reduced Height of Manometer Column.*—The quantity directly given by a manometer reading was the height of the top of the mercury column above the zero of the scale. To this must be added the height of the zero of the scale above the surface of the mercury in the reservoir. This latter quantity need not be determined with great accuracy, since by the method adopted for computing loss of head (to be explained presently) a constant added to all the observed readings does not affect the final result. Except for the necessity of applying the temperature correction, the direct readings of the scale could be used instead of the total heights of the mercury column. For the purpose of applying the corrections for temperature, the height of the zero of the scale above the surface of the mercury in the reservoir was taken as a constant, its value being measured under conditions of static pressure. The reduction to an equivalent column at 0° Cent. is made by multiplying the observed height of the column by the factor $1-0.000\,18\,T$; T being the temperature of the mercury at the time of observation, expressed in degrees Centigrade.

Strictly speaking, a correction must also be made for the rise of the surface of the mercury in the reservoir. The interior diameter of the reservoir being about sixteen times that of the glass tube, a fall of the mercury in the latter must be accompanied by a rise about $\frac{1}{256}$ as great in the former. The true shortening of the column is therefore equal to the drop of the column multiplied by $\frac{257}{256}$, or about 1.004. For the greatest loss of head measured in the entire series of experiments (about 3.4 ft.), this correction would amount to only 0.014 ft. (water).

(2) *Plotting of Reduced Readings.*—After each day's run, the reduced heights of the manometer columns and the readings of the Venturi difference-gauges were plotted as ordinates with time as abscissas. By inspection of the resulting curves it was possible to select periods during which the conditions of pressure and discharge remained steady. The data corresponding to each period were averaged, and the resulting average values of the manometer heights and difference-gauge readings were regarded as constituting an "observation." Samples of the plotted results are shown in Figs. 4 and 5 which will be referred to again.

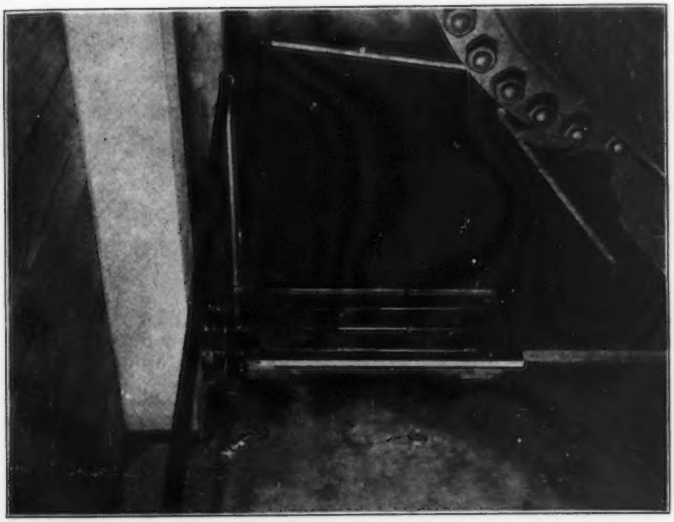

FIG. 2.

FIG 1.

(3) *Computation of Loss of Head.*—Since the pipe had the same diameter at the two pressure stations, the loss of head between these stations is found by a comparison of the pressures.

Let suffixes $(_1)$ and $(_2)$ refer to up-stream and down-stream stations respectively, and let

m_1 and m_2 denote any simultaneous heights of the mercury columns;

M_1 and M_2 values of m_1 and m_2 under static pressure;

$y = m_1 - m_2$;

$Y = M_1 - M_2$;

e = specific gravity of mercury = 13.6;

H' = loss of head between the two stations (expressed in height of water column).

Then $e(M_1 - m_1)$ = loss of head in pipe from inlet to upper station;

$e(M_2 - m_2)$ = " " " " lower "

$e[(M_2 - m_2) - (M_1 - m_1)] = e(y - Y)$ = loss of head between the two stations..... (1)

That is, $H' = e(y - Y)$..... (2)

It is evident that the value of H' is not changed by any constant error affecting all values of m_1 (including the static value), nor by a constant error affecting all values of m_2. This justifies the statement above made that great accuracy is not required in the determination of the height of the zero of the manometer scale above the surface of the mercury in the reservoir. Any constant length of the mercury column could be disregarded, except for the necessity of reducing for temperature.

(4) *Computation of Rate of Discharge.*—The theory of the Venturi meter is based upon the assumption that the velocity of flow is uniform throughout each of the cross-sections whose pressures are compared. On this assumption, and neglecting loss of head between the two sections, the rate of discharge is given by the following formula:

Let Q denote the rate of discharge in cubic feet per second, computed on the above assumption, and let the difference between the pressure-heads at points on the same level in the two cross-sections (called by Herschel "head on Venturi") be represented by H (expressed in feet of water column). Then, if a_1 and a_2 are the areas of the two cross-sections,

$$Q = \frac{a_1 \, a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gH} \quad \ldots\ldots (3)$$

If q denotes the true value of the rate of discharge,

$$q = kQ \ldots\ldots (4)$$

k being a coefficient whose value is known only by experiment.

In the Ogden meters the diameters of the two cross-sections are 54 ins. and 25.5 ins., respectively. The formula therefore becomes

$$q = 3.639\, k\sqrt{2\,g\,H}; \ldots\ldots (5)$$

q being in cubic feet per second, H in feet, g in foot-second units, and k being an abstract number.

In order to save labor in the application of the formula it is convenient to replace H by the difference in height of the two mercury columns of the difference-gauge. If h denotes this difference and e the specific gravity of mercury,

$$H = (e - 1)h; \ldots\ldots (6)$$

since the difference between the mercury heights is in part balanced by an equal and opposite difference between the heights of the water columns above the mercury. Taking 13.6 as the value of e, equation (5) may be written

$$q = 103.58\, k\sqrt{h} \ldots\ldots (7)$$

The values of k to be used with the Ogden meters were furnished to the writers, at the request of Clemens Herschel, M. Am. Soc. C. E., by the manufacturers, the Builders' Iron Foundry, Providence, R. I.

If

$$k^1 = \frac{a_1}{\sqrt{a_1^2 - a_2^2}}\, k \ldots\ldots (8)$$

the values of k^1 are given in the following table:

Head on Venturi. H (feet).	Coefficient. k^1.	Head on Venturi. H (feet).	Coefficient. k^1.
1	.995	13	.977
2	.992	14	.9765
3	.9895	15	.9755
4	.9875	16	.975
5	.986	17	.974
6	.985	18	.973
7	.9835	19	.9725
8	.9825	20	.9715
9	.9815	21	.971
10	.9805	22	.9705
11	.9795	23	.970
12	.9785		

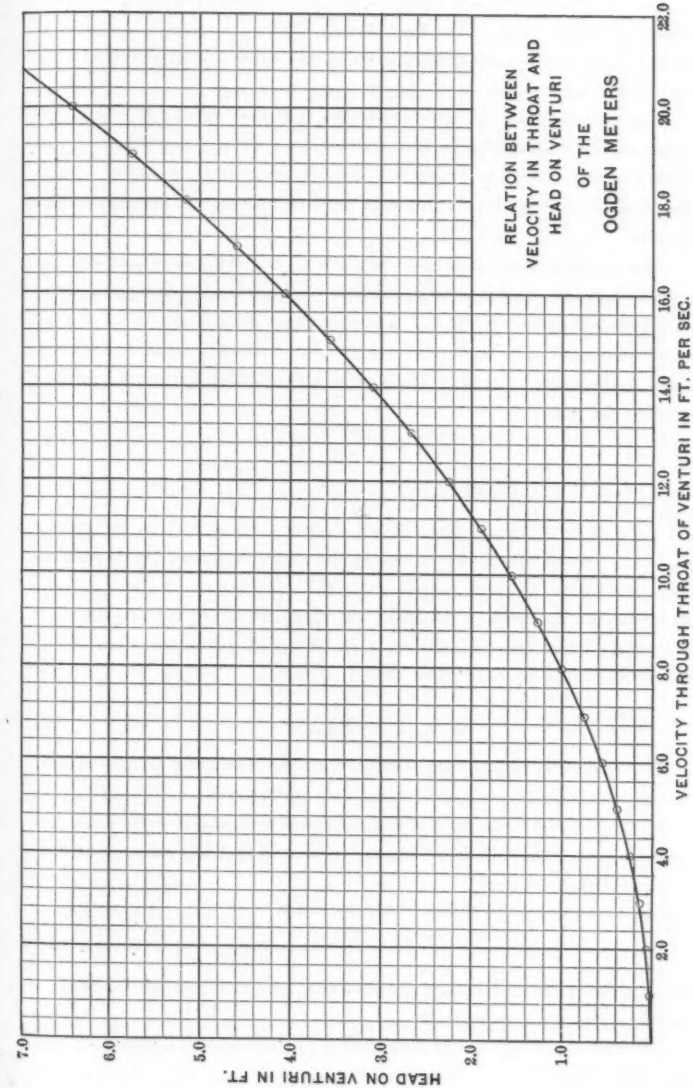
RELATION BETWEEN
VELOCITY IN THROAT AND
HEAD ON VENTURI
OF THE
OGDEN METERS
HEAD ON VENTURI IN FT.
7.0
6.0
5.0
4.0
3.0
2.0
1.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
VELOCITY THROUGH THROAT OF VENTURI IN FT. PER SEC.

FIG. 2.

These values are based upon experiments made by Herschel in 1891 with the 48-in. meter of the East Jersey Water Company; the ratio of throat diameter to main diameter being the same for this meter as for the Ogden meters.

The values of the throat velocity for different values of "head on Venturi," corresponding to the above values of the coefficient k^1, are shown in Fig. 2. This diagram is copied from one furnished by the Builders' Iron Foundry, which was used in making the actual computations.

The heights of the mercury columns of the difference-gauges were read to 0.001 ft., and it is believed that the probable error in a value of h does not exceed 0.002 ft. The corresponding error in the value of q varies with q, being less as q is greater. From the relation between q and h above given, taking $k = 1$ (which is sufficiently correct for the present purpose),

$$d\,q = \frac{5360}{q}\,d\,h. \qquad (9)$$

An error $\delta\,h = 0.002$ causes an error $\delta\,q = \frac{10.7}{q}$, which varies from 1.1 to 0.11 as q varies from 10 cu. ft. per second to 100 cu. ft. per second. The smallest values of q in the experiments under consideration were about 14 cu. ft. per second; the uncertainty of these values, so far as due to errors in the observed values of h, is probably about 5 per cent. The observational error in values of q greater than 35 cu. ft. per second is probably less than 1 per cent.

General Results.

In order to facilitate comparison with the results of other experiments, the results are represented in three ways:

(1) The loss of head per 1 000 ft. has been computed from each observation, and the relation between this loss and the velocity of flow represented graphically.

(2) The values of c in the Chezy formula,

$$v = c\sqrt{r\,s}. \qquad (10)$$

has been computed for each observation, and the relation between v and c represented graphically.

(3) The value of f in the formula for loss of head in a pipe of diameter d and length l,

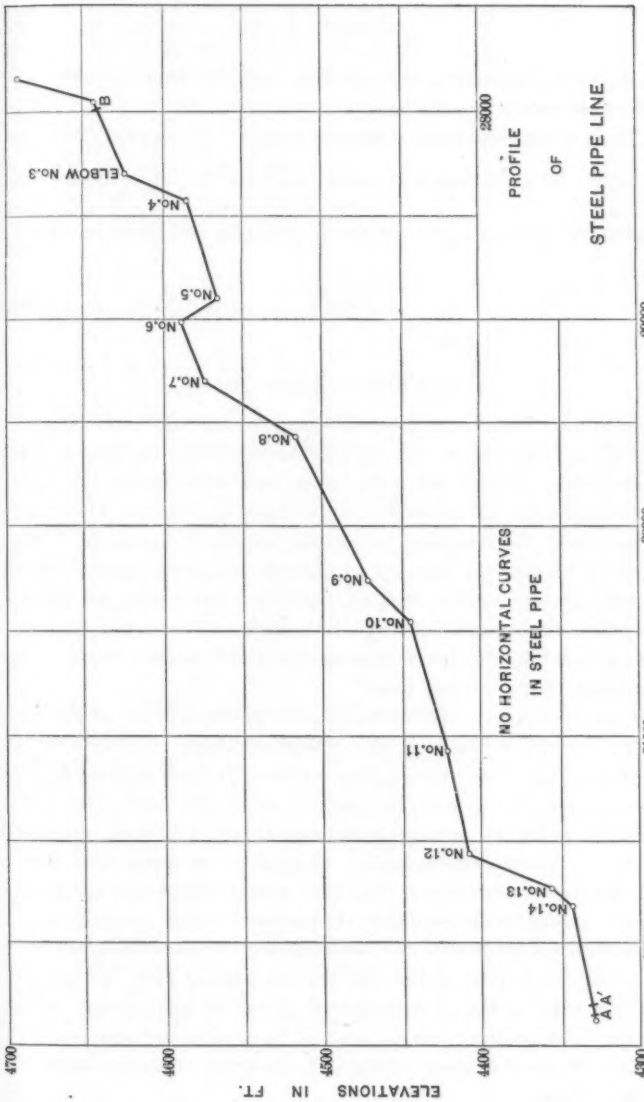
PROFILE
OF
STEEL PIPE LINE
NO HORIZONTAL CURVES
IN STEEL PIPE
B
ELBOW No.3
No.4
No.5
No.6
No.7
No.8
No.9
No.10
No.11
No.12
No.13
No.14
A'
A
ELEVATIONS IN FT.
4700
4600
4500
4400
4300
HORIZONTAL DISTANCES IN FT.
32900
31000
30000
29000
28000

FIG. 8.

$$H' = f \frac{l}{d} \frac{v^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

has been computed for each observation, and the relation between f and v represented by a diagram.

There is a definite relation between c and f. In the Chezy formula the value of s, the hydraulic slope, is $\frac{H'}{l}$, and the value of the hydraulic radius r is $\frac{d}{4}$. The above two formulas are therefore identical if

$$f = \frac{8g}{c^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

Loss of Head in Venturi Meter.

As already stated, the difference-gauges attached to the Venturi meters were designed to show the differences between the pressures at three sections: (1) just before the contraction of the stream, the diameter being 54 ins.; (2) at the throat, the diameter being 25.5 ins., and (3) just below the expanding stream, the diameter being 54 ins. The difference between the pressures at the first and third sections shows the loss of head caused by friction, and by the contraction and expansion of the stream in passing through the meter. The relation between this loss and the rate of flow through the meter is represented both graphically and in tabular form.

Some trouble was experienced from the presence of dirt at the top of the mercury columns of the difference-gauges. It fortunately happened that this difficulty was confined, in both gauges, to the tube communicating with the section below the meter (the tube marked C in Fig. 1*b*), so that the estimate of rate of discharge was not affected. The observations for loss of head in the meter were, however, rendered less accurate than they would otherwise have been. The uncertainty in the reading of the mercury column probably in no case exceeded 0.004 ft., but this was sufficient to render valueless certain of the observations taken with low velocities of flow. In spite of this, the entire series of observations serves to show pretty satisfactorily the relation between loss of head in meter and velocity of flow, within the range of the experiments. The discussion of results follows.

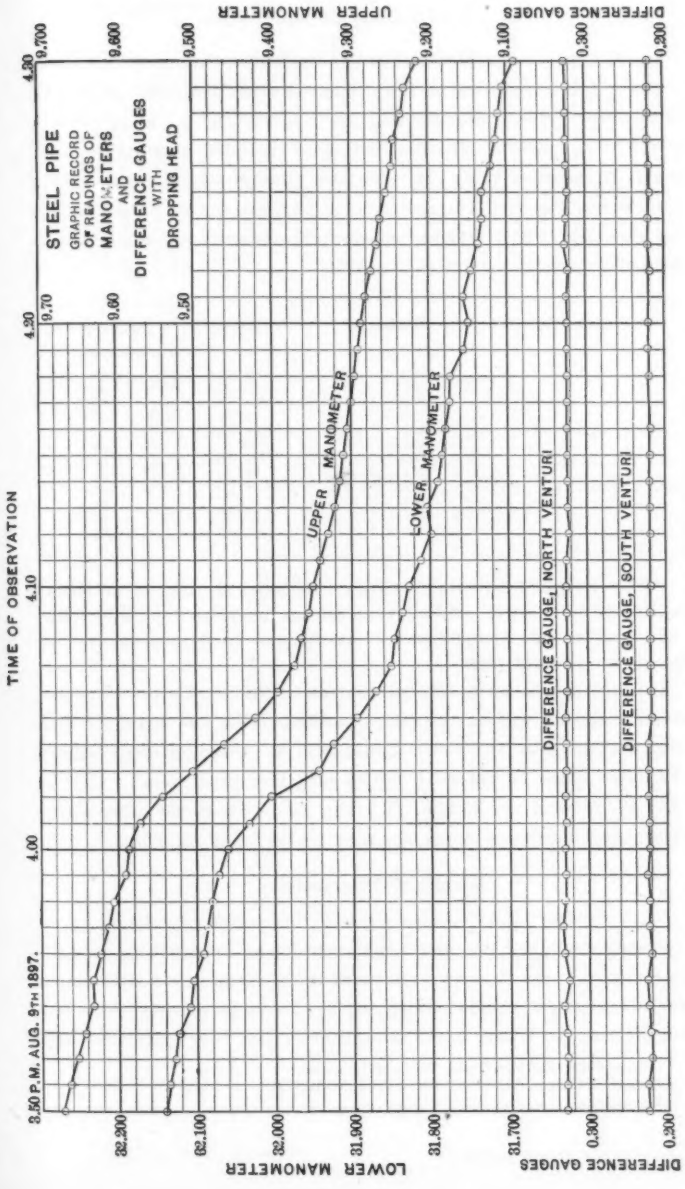
STEEL PIPE
GRAPHIC RECORD
OF READINGS OF
MANOMETERS
AND
DIFFERENCE GAUGES
WITH
DROPPING HEAD
TIME OF OBSERVATION
3.50 P.M. AUG. 9TH 1897.
4.00
4.10
4.20
4.30
LOWER MANOMETER
UPPER MANOMETER
DIFFERENCE GAUGES
DIFFERENCE GAUGE, NORTH VENTURI
DIFFERENCE GAUGE, SOUTH VENTURI

Fig. 4.

II.—Experiments on Steel Pipe.

Description of Pipe Line.

For a full description of the dimensions and construction of the riveted steel pipe, reference may be made to the paper previously cited.* The portion experimented upon included nearly the entire length of the steel pipe. In plan this pipe is straight; the profile is shown in Fig. 3. The positions of vertical curves or elbows are also shown in Fig. 3, the radius of curvature and total angle in each case being given in the following table:

Elbow No.	Station.	Angle.	Radius in feet.
3	283	13° 30′	30
4	284 + 80	14° 23′	30
5	289	14° 8′	30
6	290 + 10	14° 46′	30
7	293	8° 58′	30
8	295 + 70	8° 8′	30
9	302 + 50	3° 20′	30
10	304 + 50	4° 49′	30
11	310 + 70	1° 0′	30
12	315 + 81.6	17° 15′	40
13	316 + 71.6	13° 39′	30
14	318 + 31.6	7° 25′	30

"The change of direction between successive sections of an elbow was limited to 4° or 5°, so that each section is about 2½ or 3 ft. long. * * * The butt straps, sizes of rivets and rivet spacing are the same as for straight pipe."†

Throughout the series of observations on the steel pipe the upper pressure station was at the point marked *B* on the profile. The lower station, near the power house, was at *A* during observations 1–12, after which it was at *A'*. The length of pipe from *A* to *B* is 4 427 ft.; from *A'* to *B*, 4 367 ft. The diameter of the pipe varies slightly in different portions, the mean value being 72.22 ins.‡

Record of Observations.

(*1*) *Examples of Full Data Upon Which Tabulated Results Are Based.*—As already explained, the quantities recorded as constituting a single "observation" are in every case based upon readings taken at short intervals during a period of some length. These periods were selected after plotting, to a time base, the values of the four quantities ob-

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.
† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 262.
‡ *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 259.

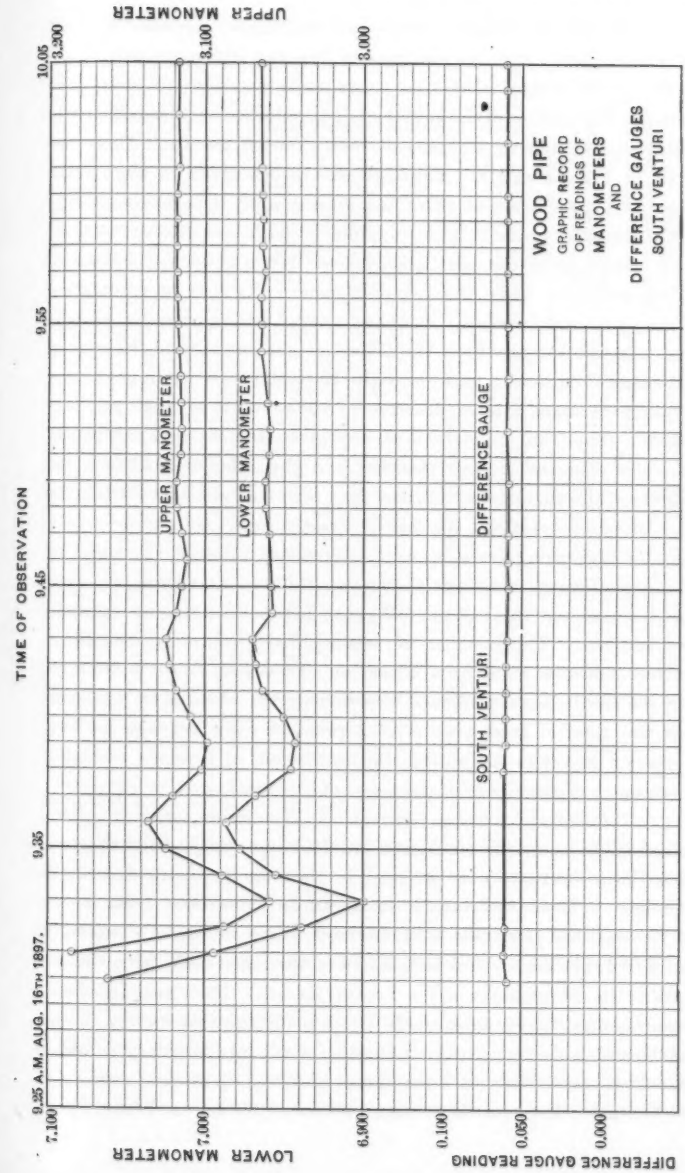
WOOD PIPE
GRAPHIC RECORD
OF READINGS OF
MANOMETERS
AND
DIFFERENCE GAUGES
SOUTH VENTURI
UPPER MANOMETER
LOWER MANOMETER
DIFFERENCE GAUGE
SOUTH VENTURI
TIME OF OBSERVATION
9.25 A.M. AUG. 16TH 1897.
9.35
9.45
9.55
10.05
UPPER MANOMETER
3.200
3.100
3.000
LOWER MANOMETER
7.100
7.000
6.000
DIFFERENCE GAUGE READING
0.100
0.050
0.000

Fig. 5.

served, namely, the heights of the two manometer columns (reduced for temperature), and the readings of the two difference-gauges attached to the Venturi meters (at times, however, only one meter was in use). The nature of the actual records obtained will be sufficiently indicated by showing samples of these plotted results. These are given in Figs. 4 and 5. Although the record given in Fig. 5 was taken during an experiment on the wooden pipe, it is representative of the character of the records obtained in both sets of observations, in all cases except the nine observations taken under conditions of falling pressure; while the record shown in Fig. 4 is representative of these nine observations. In these cases of falling pressure, periods were selected during which the difference-gauges showed the rate of discharge to be nearly uniform, while the manometer readings showed a nearly constant rate of decrease of the pressure at each pressure station. This rate of decrease at any given instant had practically the same value at the two stations, and the difference between the heights of the manometer columns thus remained constant during the time of an observation. In case of observations made under conditions of decreasing pressure, any discrepancy between the times of taking manometer readings at the two stations introduces an error into the observed value of the difference between heights of manometer columns. The instants of reading were as nearly coincident as they could be made by comparison of watches, and it is believed that the results are not affected by serious error. Assuming, as above, that the time of transmission of a pressure-wave between the two stations is less than one second, the oscillations of pressure furnish a check on the coincidence of the times of observation.

(2) *Tabulation of Reduced Observations.*—The entire series of experiments made upon the steel pipe resulted in twenty-nine "observations" whose periods were selected in the manner above described. The dates of these observations are shown in Tables Nos. 1 and 2.

The date and period of each observation are shown in columns 2 and 3 of Table No. 1. Columns 4 to 9 of the same table give the results of the observations on the Venturi meters. The "reading of difference-gauge," given in columns 4 and 6 for the two meters, is the difference between the observed heights of the two mercury columns connected respectively with the throat and the up-stream section of the Venturi. The equivalent water column ("head on Venturi") is

TABLE No. 1.—STEEL PIPE DISCHARGE OBSERVATIONS.

1	2	3	4	5	6	7	8	9
			NORTH VENTURI METER.		SOUTH VENTURI METER.			
No.	Date.	Time.	Reading of difference-gauge. (Feet.)	Rate of discharge (cubic feet per second).	Reading of difference-gauge. (Feet.)	Rate of discharge (cubic feet per second).	Rate of discharge of main pipe (cubic feet per second).	Mean velocity of flow in main pipe (feet per second).
	1897.							
1....	Aug. 4	2.35— 2.45	.....	.....	0.022	14.5	14.5	0.510
2....	" 4	3.22— 3.31	.....	.....	.199	44.8	44.8	1.575
3....	" 6	2.10— 3.00	0.022	14.5	.....	.....	14.5	0.510
4....	" 6	3.15— 3.40	.093	30.9	.....	.....	30.9	1.086
5....	" 6	3.50— 4.15	.090	30.3	.....	.....	30.3	1.065
6....	" 7	1.33— 1.56	.....	.....	.225	47.5	47.5	1.670
7....	" 7	1.57— 2.15	.....	.....	.225	47.5	47.5	1.670
8....	" 7	2.45— 2.53	.....	.....	.512	71.2	71.2	2.503
9....	" 7	3.15— 3.29	.002	4.6*	.481	69.2	73.8*	2.594*
10....	" 7	4.11— 4.23	.181	42.6	.391	62.5	105.3	3.701
11....	" 7	5.00— 5.13	.078	28.2	.141	37.6	65.8	2.313
12....	" 7	5.15— 5.50	.077	28.1	.140	37.5	65.6	2.306
13....	" 9	1.42— 2.15	.127	35.8	.296	54.5	90.3	3.174
14....	" 9	2.36— 3.01	.090	30.3	.226	47.5	77.8	2.735
15....	" 9	3.26— 4.00	.333	57.8	.221	47.1	104.9	3.687
16....	" 9	4.12— 4.30	.323	57.0	.219	46.9	103.9	3.652
17....	" 11	8.40— 8.51	.021	14.1	.....	.....	14.1	0.496
18....	" 11	8.55— 9.15	.021	14.1	.....	.....	14.1	0.496
19....	" 11	9.30—10.00	.099	31.7	.....	.....	31.7	1.114
20....	" 11	10.39—10.43	.095	31.2	.043	21.0	52.2	1.835
21....	" 11	11.10—11.31	.086	29.6	.159	40.0	69.6	2.447
22....	" 11	11.35—12.00	.087	29.8	.160	40.1	69.9	2.457
23....	" 12	9.15— 9.28	.204	45.5	.....	.....	45.5	1.599
24....	" 12	9.29— 9.39	.205	45.7	.....	.....	45.7	1.606
25....	" 13	8.30— 8.50	.....	.....	.026	16.0	16.0	0.562
26....	" 13	9.13— 9.40	.111	33.7	.072	27.2	60.9	2.141
27....	" 13	10.07—10.40	.270	51.8	.123	35.3	87.1	3.062
28....	" 13	11.36—11.46	.401	63.2	.211	46.2	109.4	3.846
29....	" 13	11.48—12.03	.400	63.1	.210	46.1	109.2	3.839

* The rate of the discharge of the N. meter was so small during this observation that it could not be accurately measured.

found by multiplying the tabulated number by 12.6 or $e - 1$, e being the specific gravity of mercury. The values of the head on Venturi are not given in Table No. 1, but will be found in Tables Nos. 8 and 9. Columns 5, 7, 8 and 9 need no explanation. In computing values of the mean velocity in the main pipe, the diameter has been taken as 72.22 ins.

The manometer observations, with the values of the loss of head deduced from them, are recorded in Table No. 2. The headings of the columns explain sufficiently the meanings of the quantities tabulated. Column 2 gives the length of the main pipe between the points of at-

TABLE No. 2.—Steel Pipe.—Manometer Observations.

1	2	3	4	5	6	7	8	9
No.	Length of pipe. (Feet.)	Heights of Manometer Columns Reduced to 0° Cent. (Feet.)			Static difference of mercury columns reduced to 0° cent. (Feet.)	Loss of Head between Manometers.		Loss of head per 1 000 ft. (in feet of water).
		Upper.	Lower.	Difference.		Mercury (Feet.)	Water. (Feet.)	
1..	4 427	11.251	33.916	22.665	22.668	0.003	0.041	0.0092
2..	4 427	11.059	33.686	22.627	22.668	.041	.558	.126
3..	4 427	11.258	33.914	22.656	22.668	.012	.163	.0369
4..	4 427	11.201	33.847	22.646	22.668	.022	.299	.0676
5..	4 427	11.204	33.847	22.643	22.668	.025	.340	.0768
6..	4 427	11.018	33.661	22.643	22.698	.055	.748	.169
7..	4 427	11.013	33.659	22.646	22.698	.052	.707	.160
8..	4 427	10.590	33.171	22.581	22.698	.117	1.591	.359
9..	4 427	10.559	33.136	22.577	22.698	.121	1.646	.372
10..	4 427	Dropping	Dropping	22.453	22.698	.245	3.332	.753
11..	4 427	10.720	33.300	22.580	22.698	.118	1.605	.363
12..	4 427	10.733	33.316	22.583	22.698	.115	1.564	.353
13..	4 367	10.296	32.816	22.520	22.707	.187	2.543	.582
14..	4 367	10.559	33.113	22.554	22.707	.153	2.081	.477
15..	4 367	Dropping	Dropping	22.472	22.707	.235	3.196	.732
16..	4 367	"	"	22.472	22.707	.235	3.196	.732
17..	4 367	11.260	33.960	22.700	22.707	.007	.095	.0218
18..	4 367	11.262	33.966	22.704	22.707	.003	.041	.0093
19..	4 367	11.207	33.896	22.689	22.707	.018	.245	.0560
20..	4 367	11.022	33.669	22.647	22.707	.060	.816	.187
21..	4 367	10.754	33.352	22.598	22.707	.109	1.482	.340
22..	4 367	10.748	33.342	22.594	22.707	.113	1.537	.352
23..	4 367	11.080	33.752	22.672	22.707	.035	.476	.109
24..	4 367	11.086	33.750	22.664	22.707	.043	.585	.134
25..	4 367	11.261	33.963	22.702	22.707	.005	.068	.0156
26..	4 367	10.890	33.520	22.630	22.707	.077	1.047	.240
27..	4 367	10.375	32.923	22.548	22.707	.159	2.162	.495
28..	4 367	Dropping	Dropping	22.458	22.707	.249	3.386	.776
29..	4 367	"	"	22.460	22.707	.247	3.359	.769

tachment of the two manometers. The point of attachment of the upper manometer remained unchanged during the experiments; that of the lower was changed after observation No. 12.* Column 6 gives the difference between the heights of the two manometer columns under static conditions. The observed value of this difference changed because the quantity of mercury in the lower manometer changed, for reasons to be considered presently. The numbers in column 7 are obtained from those in columns 5 and 6 by subtraction. The values in column 7 are reduced to equivalent water heights (column 8) by multiplying by 13.6; and the loss of head per thousand feet (column 9) is found from total loss of head (column 8) and length of pipe (column 2).

*The reason for this change was a suggestion that the point of attachment was at first too near the fork in the main pipe. The results, however, show no evidence of vitiation from this cause.

TABLE No. 3.—STEEL PIPE. GENERAL RESULTS.

1	2	3	4	5
No.	Mean velocity in main pipe. Feet per second.	Loss of head per 1 000 feet. (Feet.)	Value of c in formula $v = c\sqrt{rs}$.	Value of f in formula $H^1 = f \cdot \frac{l}{d} \frac{v^2}{2g}$.
17	.496	0.0218	87	0.0343
18	.496	.0093	132	.0147
1	.510	.0092	137	.0137
3	.510	.0369	68	.0549
25	.562	.0156	116	.0191
5	1.065	.0768	99	.0262
4	1.086	.0676	108	.0222
19	1.114	.0560	121	.0174
2	1.575	.126	114	.0197
23	1.599	.109	125	.0165
24	1.606	.134	113	.0201
6	1.670	.169	105	.0235
7	1.670	.160	108	.0222
20	1.835	.187	109	.0215
26	2.141	.240	113	.0202
12	2.306	.353	100	.0257
11	2.313	.363	99	.0262
21	2.447	.340	108	.0219
22	2.457	.352	107	.0226
8	2.503	.359	108	.0222
9	2.594	.372	110	.0214
14	2.735	.477	102	.0246
27	3.062	.495	112	.0204
13	3.174	.582	107	.0224
16	3.652	.732	110	.0212
15	3.687	.732	111	.0208
10	3.701	.753	110	.0213
29	3.839	.769	113	.0202
28	3.846	.776	113	.0203

(3) *General Results.*—In Table No. 3 are shown three sets of results, arranged for convenient comparison with other recorded experiments and accepted results. The observations are arranged according to values of v, the mean velocity of flow (column 2), and in columns 3, 4 and 5 are given the values of the three quantities most commonly used in estimating the carrying capacity of pipes. The quantities given in this table are also represented graphically in Figs. 6, 7 and 8.

Fig. 6 shows the loss of head per thousand feet as a function of the mean velocity of flow. The curve expressing this relation would be a parabola with vertex at the origin, and principal axis parallel to the ordinates representing loss of head, if c and f were constants.

In Fig. 7 each observation is represented by a point whose abscissa is v, and whose ordinate is the value of c in the Chezy formula, $v = c\sqrt{rs}$. In like manner Fig. 8 shows, for each observation, the value of v and that of f, the coefficient in the formula for loss of head,

$$H' = f \frac{l}{d} \frac{v^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

From an inspection of Figs. 6, 7 and 8 it is evident that the observations do not clearly disclose any law of variation of the coefficients c and f with the velocity of flow. The curves which appear to best represent the distribution of the plotted points are $A\,B$ (Fig. 7) and $A'B'$ (Fig. 8). [It is to be noticed that the value of f from observation No. 3 (0.0549) is so great as to throw the point beyond the limit of Fig. 8.] These curves show, between $v = 1.5$, and $v = 2.5$, a slight decrease in c and increase in f; and from $v = 2.5$ to $v = 4.0$, a slight increase in c and decrease in f. Such a law of variation cannot, however, be regarded as well established by the results shown.

It is apparent that, for values of v less than 4 ft. per second, the values of c and f do not greatly vary with the velocity. The average of all the values of c given in Table No. 3 is 109.1; omitting cases in which v is less than 1 ft. per second, the average is 109.4. The corresponding values of f are 0.0216 and 0.0215.

In addition to the above quantities, it may be well to record the values of n, the coefficient of roughness in Kutter's formula. The above mean value of c gives for n values ranging from about 0.013 to 0.015, the smaller values corresponding to low velocities of flow and the larger to the higher velocities.

Degree of Reliability of Results.

The reliability of the measurement of rate of discharge has already been discussed.

In the determination of loss of head in the steel pipe a difficulty was met which somewhat impaired the reliability of the results. This was the difficulty of making the manometer reservoir and the pipe containing the mercury column perfectly tight under the high pressure existing at the lower manometer station. After the experimenting had continued for several days, it was found that there had been a leakage of mercury sufficient to appreciably affect the manometer readings. As the fluctuations of the surface of the mercury in the reservoir could not be accurately measured, the changes in the quantity of mercury in the manometer introduced errors into the observed values of y (the difference between heights of mercury columns). It has already been pointed out that a constant error in these values is of no consequence; but the leakage of mercury introduced a variable error of uncertain amount. Had the difficulty been foreseen, means

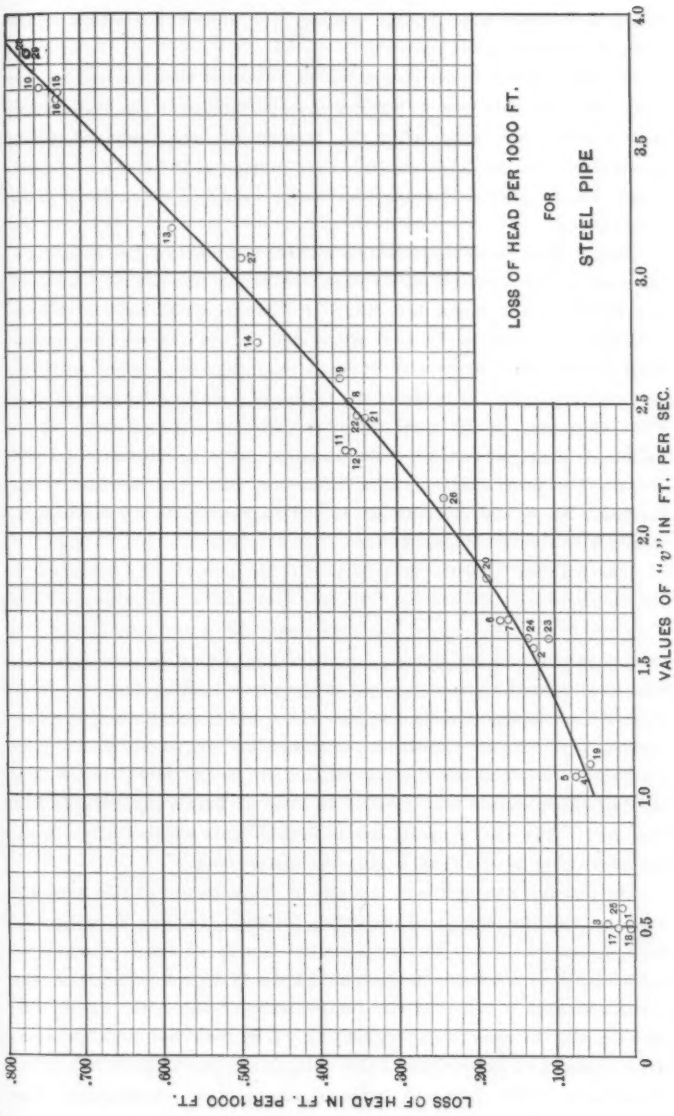
LOSS OF HEAD PER 1000 FT.
FOR
STEEL PIPE
LOSS OF HEAD IN FT. PER 1000 FT.
.800
.700
.600
.500
.400
.300
.200
.100
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.

FIG. 6.

would have been provided for reading accurately the position of the mercury surface in the reservoir.

A careful study of the results made it evident that the leakage was very slow, the only case in which a marked change in the static readings was noticed being after the manometer had remained under pressure over night. During the latter portion of the series of experiments, the leakage apparently decreased so much as to be of little importance.

Had it been possible to secure reliable readings of the heights of the manometer columns, under static conditions on each day on which observations were made, the effect of leakage could have been estimated and the errors due to it eliminated. This was not always possible, but a sufficient number of static readings was obtained so that a fair estimate could be made of the values of "static difference of mercury columns" to be used with the different observations. These values, as nearly as could be estimated, are given in column 6 of Table No. 2. The value of the "static difference" shows an increase at observation No. 6, and again at No. 13. This was due to the addition of mercury to the lower manometer.

It is believed that the greater part of the irregularity observed in the values of loss of head per thousand feet, and in the values of c and f, is due to the leakage of mercury from the lower manometer. A further possible cause of error, especially in the earlier observations, may have been the presence of air in the pipes leading from the main pipe to the manometers. In the case of the lower manometer, the connecting pipe was necessarily of considerable length, since the only practicable location for the manometer was in the power-house, and careful manipulation was needed to keep so long a pipe wholly free from air. This was probably accomplished more successfully during the later observations than during the earlier ones.

The possible errors due to changes of temperature of the mercury columns, and the method of correcting for such changes, have already been discussed. As a further possible source of error, changes of temperature of the water in the main pipe may be mentioned. It was not practicable to observe the temperature of the water within the pipe during the time of experimenting. The best that could be done was to take the temperature of the water in the waste flume below the power-house. No complete series of temperature observations was

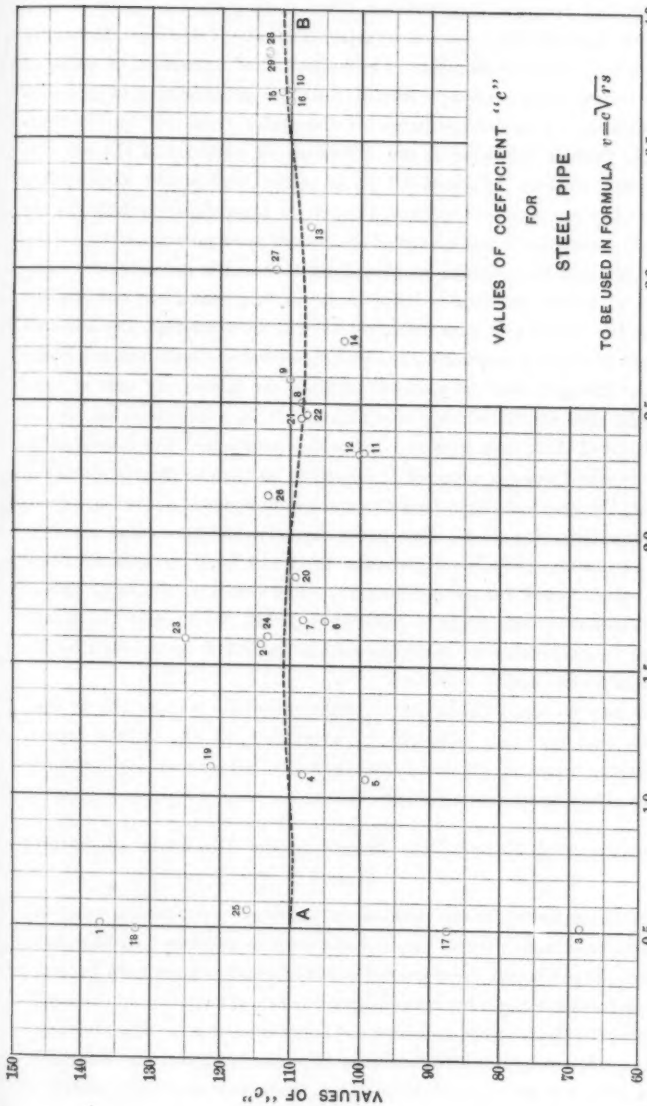
VALUES OF COEFFICIENT "c"
FOR
STEEL PIPE
TO BE USED IN FORMULA $v = c\sqrt{rs}$
A
B
VALUES OF "c"
VALUES OF "v" IN FT. PER SEC.
150
140
130
120
110
100
90
80
70
60
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

FIG. 7.

taken, but frequent observations, taken during the time from August 4th to August 11th, showed a variation of about 2° Cent., the range being from 16 to 18 degrees. The coefficient of expansion of water at this temperature is about 0.000174, nearly as great as that of mercury. A variation of the temperature of the water from 16° to 18° Cent. would change the value of the difference of pressure at the two manometer stations by about 0.1 ft. of water, and would appreciably affect the manometer readings, though its importance would not be great, except for small values of the loss of head measured.

Although the plotted results (Figs. 6, 7 and 8) indicate the presence of errors sufficiently large to be of importance in comparison with the quantities measured, yet it will be seen that the absolute values of the errors affecting the pressure measurements are not great. It was thought that the method adopted for measuring loss of head should give results reliable to within 0.1 ft. in absolute value, and it is believed that, even with the difficulties mentioned, the results actually reached possess a degree of reliability as high as the standard set. Thus, the greatest discrepancy in the whole series is shown by observations Nos. 1 and 3. The mean velocity has the same value in the two cases, while No. 1 gives the least and No. 3 the greatest of all the values found for the coefficient c. The values of the total loss of head differ by only 0.122 ft. (column 8, Table No. 2), and the actual error in each value is doubtless within the 0.1 ft. mentioned as the standard of accuracy expected.

It may be noted further, that, if the value of c is regarded as constant, and its value obtained by averaging all the values found, excluding cases in which v is less than 1 ft. per second, the probable error of this mean value is about 0.9.

Comparison of Known Data Regarding the Discharge of Riveted Pipes.

So far as known to the authors, no previous experiments upon the discharge of new riveted pipes of diameter as great as 6 ft. have been made. Experiments upon riveted pipes of smaller sizes have been too few to furnish a basis for establishing the law of variation of the coefficient c with the velocity, diameter and other conditions. The resistance to flow offered by the pipe doubtless depends upon various elements—thickness of plates, length of sections, frequency of rivets and size of

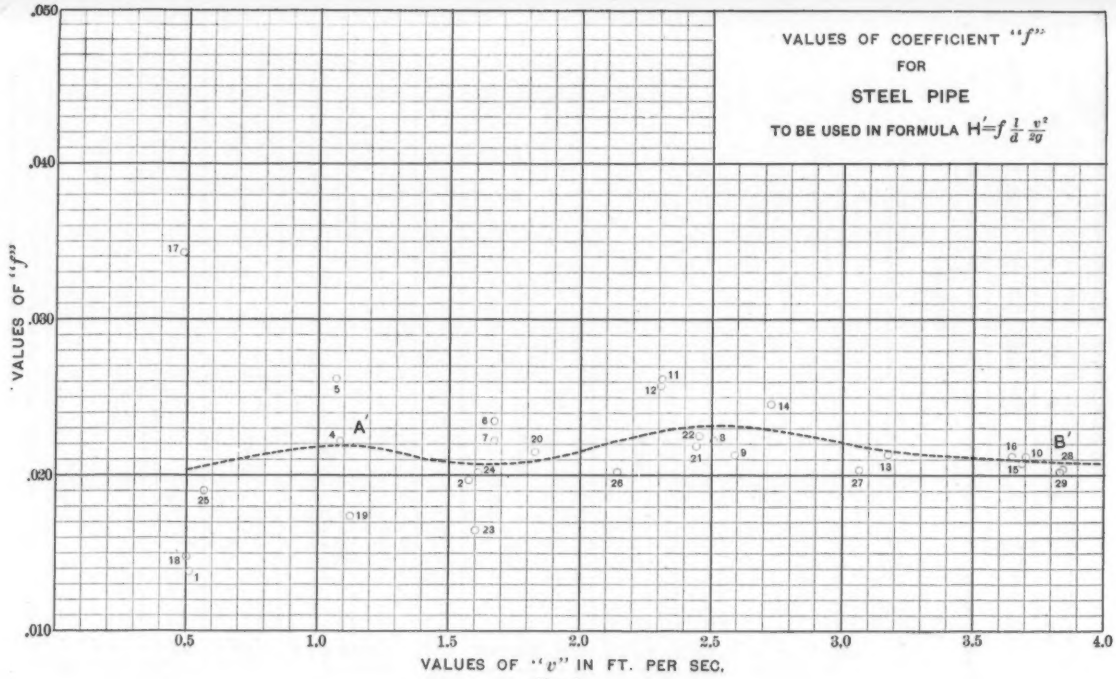
VALUES OF COEFFICIENT "f"
FOR
STEEL PIPE
TO BE USED IN FORMULA $H'=f\frac{l}{d}\frac{v^2}{2g}$
VALUES OF "f"
.050
.040
.030
.020
.010
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.
A'
B'

FIG. 8.

TABLE No. 4.—Experiments on Riveted Pipes,

No.	1	2	3	4	5	6	7	8	9	10	11	12
Age.	5 yrs.	New.	5 yrs.	5 yrs.	New.	1 yr.	14 yrs.	14 yrs.	5 yrs.	New.	New.	14 yrs.
Diam. (ins.)...	11	11.25	13	15	16	17	24	24	26	33	35	36
Vel.	Coefficient c.											
0.5												
1.0												
1.3		101.3										
1.48												80.3
1.5												
2.0												
2.44												
2.5												
2.78		114.0										
3.0												
3.23												
3.27												
3.32							75.8	78.5				
3.5												
3.52											126.8	
3.87			121.6									
3.90												
3.96										123.2		
4.0												
4.38				111.6								
4.5												
4.58					112.3							
4.60			109.4									
4.71	107.1											
4.90		122.5										
4.93												
5.0												
5.5												
6.0												
6.09	110.6											
6.67		126.5										
6.84				117.8								
6.93	111.5											
6.96			113.4									
7.31				119.1								
8.46				119.0								
8.65			113.0									
8.66	113.4											
8.85		128.1										
10.03	115.5											
10.53		129.9										
10.59				121.6								
10.71			114.4									
12.09				121.3								
12.60									134.1			
20.14						131.1						
n	.010	.010	.011	.011	.011	.010	.017	.016	.010	.011	.011	.01

WITH DIAMETERS RANGING FROM 11 INS. TO 103 INS.

No.	13	13a	14	15	16	17	18	19	19a	19b	20	21	22
Age.	New.	4 yrs.	New.	New.	New.	New.	New.	New.	4 yrs.	4 yrs.	New.	New.	5 yrs.
Diam. (ins.)	36	36	38	38	42	42	42	48	48	48	48	72	103
Vel.	Coefficient c.												
0.5												110	126.5
1.0	86.0					96.0	101.0	101.2	78.0	97.2	97.1	110	116.6
1.3													
1.48													
1.5	90.6					103.0	102.8	105.4	84.6	100.8	98.7	111	112.7
2.0	95.2					107.9	104.3	108.8	89.6	103.3	100.3	110	110.3
2.44					115.9								
2.5	99.4					111.0	105.5	111.2	92.4	104.9	101.6	108	108.8
2.78													
3.0	103.3					112.6	106.4	112.8	93.0	105.3	102.2	108	107.7
3.23			114.0										
3.27			116.6										
3.32													
3.5	107.0					113.0	107.2	113.4	93.2	104.8	103.6	110	106.9
3.52													
3.87													
3.90				109.2									
3.96													
4.0	110.6					112.8	107.8	113.2	94.0	104.0	104.2	111	106.2
4.38													
4.5	114.0					111.8	108.2	112.4	94.2	103.7	104.7		105.6
4.58													
4.60													
4.71													
4.90													
4.93		106.3											
5.0	117.2					110.8	108.4	112.0	94.4	103.7	105.1		
5.5	120.4					110.2	108.5	111.7	94.7	103.7	105.2		
6.0	123.6					110.0	108.5	111.6	94.9	103.7	105.2		
6.09													
6.67													
6.84													
6.93													
6.96													
7.31													
8.46													
8.65													
8.66													
8.85													
10.02													
10.52													
10.59													
10.71													
12.09													
12.60													
20.14													
n	.014	.013	.013	.013	.013	.013	.013	.013	.016	.014	.014	.014	.015

rivet-heads, nature of interior coating, and the number, frequency and sharpness of bends or elbows. These various conditions differ so much in the pipes hitherto experimented upon that hardly any two sets of results are directly comparable. The available data, so far as known to the authors, for pipes not less than 11 ins. in diameter, are shown in Table No. 4. The values of c are arranged according to values of the diameter, and also of the mean velocity of flow. At the bottom of each vertical column is given the value of n, the Kutter coefficient of roughness, as computed from the corresponding series of experiments. These values have not been computed with great precision, being taken from Kutter's diagram; and in case of experiments covering a considerable range of velocities, the recorded value of n is intermediate between the extreme values given by the experiments. Additional particulars regarding the different series of experiments from which the tabular results are deduced will now be given.

1, 3 and 4. Experiments by Hamilton Smith, M. Am. Soc. C. E., at North Bloomfield, Cal., in October, 1876. Pipes of sheet iron, single riveted, made in lengths of about 20 ft., "stove-pipe" or taper joints. Coated by immersion in boiling asphaltum and coal tar. In use 5 years, but interior surfaces worn smooth. Length in every case about 700 ft. Discharge measured by weirs. Loss of head by leveling between surface of supply reservoir and outflowing stream, correcting for velocity head. Smith's "Hydraulics," p. 302.

2. Experiments by Darcy, 1849–51. New sheet-iron pipe, coated with bitumen; screw joints; length, 365.5 ft. Discharge determined by measurement in tanks; loss of head (probably) either by water piezometers or mercury gauges. The tabulated results, in English units, are taken from Smith's "Hydraulics," p. 226.

5. Experiment by Arthur L. Adams, M. Am. Soc. C. E., on conduit of Astoria, Ore., Water-Works. Steel pipe, cylinder joints; 16 ins. is diameter of small sections; asphalt coating. Discharge measured by rise of reservoir surface in test of 18 hours; loss of head by open stand-pipes. The measurement of loss of head for steel pipe was indirect; the total loss in the combined lengths of 19 130 ft. of stave pipe and 16 416 ft. of steel pipe was measured, and at the same time the loss in 4 188 ft. of stave pipe alone. *Trans. Am. Soc. C. E.*, Vol. xxxvi, p. 21.

6. Experiment by Hamilton Smith at Texas Creek, Cal. Pipe of sheet iron, made in lengths of 20 ft.; heavy coating of asphaltum and coal tar. Length, 4 439 ft. Discharge measured by weir, 5.5 ft. long; loss of head by leveling between surfaces of supply and discharge tanks. Smith's "Hydraulics," p. 311.

7. Experiment by George W. Rafter, M. Am. Soc. C. E., on old conduit of Rochester, N. Y., Water Works, July and August, 1890. Cylinder joints. Length of experimental section, 1 901 ft. Discharge determined by rise of reservoir surface; loss of head by water piezometers, *Trans. Am. Soc. C. E.*, Vol. xxvi, p. 20.

The tabulated results are based upon a discharge of 6 742 000 galls. per day. Four gaugings of this conduit, made when it was new (in 1876), gave as an average 8 704 000 galls. per day.* Assuming equal ratio of decrease of capacity to have taken place in all parts of the conduit, the original gaugings would give about 97 as the value of c (for $v = 4.25$) when the pipe was new.

8. Same experiment as No. 7, but a different portion of the conduit. Length, 10 541 ft. Loss of head measured by Bourdon gauges.

9. Experiment by Hamilton Smith on "Humbug" pipe of North Bloomfield (Cal.) Mining Company. Pipe of sheet iron, coated with asphaltum and coal tar. Laid in 1868; experiment made in 1873, the surface being then quite smooth. Measurement of discharge not regarded as very exact. Loss of head measured by leveling between surfaces of supply and discharge reservoirs. Length, 1 194 ft. Smith's "Hydraulics," p. 309.

10. Experiment by Isaac W. Smith, M. Am. Soc. C. E., on conduit of Portland (Ore.) Water-Works. Cylinder joints; asphalt coating. Observations made on February 27th, 1896. Discharge (probably) measured by weir; loss of head by water piezometers. Length, 34 176 ft. *Trans. Am. Soc. C. E.*, Vol. xxvi, p. 203.

11. Same experiment as No. 10; another portion of conduit. Length, 39 809 ft.

12. Same experiment as Nos. 7 and 8; another portion of conduit. Loss of head measured by water piezometers. Length, 50 819 ft.

The mean of the above-mentioned four measurements made in 1876 would give, for this section of pipe, a value of c of about 103 for $v = 1.9$; assuming equal ratio of decrease of capacity for all parts of the conduit.

13. Experiments by Clemens Herschel, M. Am. Soc. C. E., on conduit of the East Jersey Water Company, from Belleville to South Orange Avenue, 1892. Cylinder joints, asphalt coating. Discharge measured by "imperfect weir;" loss of head by Bourdon gauges. Length, 25 000 ft. Herschel, "115 Experiments on the Carrying Capacity of Large, Riveted, Metal Conduits," pp. 28, 52.

13*a*. Same conduit as No. 13; experiment by Herschel in 1896. Discharge measured by Venturi meter; loss of head by Bourdon gauges. Length, 24 720 ft. "115 Experiments," pp. 28, 52.

14. Experiments by Emil Kuichling, M. Am. Soc. C. E., on new conduit of Rochester Water-Works between Hemlock Lake and Rush

* "Annual Reports Executive Board, Rochester, N. Y." 1894-95, p. 169.

Reservoir, October and December, 1895. Cylinder joints, inner sections 38 ins. in diameter; part coated with asphalt, part with Sabin's japan. Length, 91 641 ft. Discharge measured by rise of reservoir surface; loss of head by reading mercury gauge at lower end and observing level of water in feeding chamber at upper end. "Annual Reports Executive Board, Rochester, N. Y.," 1894–95, p. 344.

A portion of this conduit (from West Bloomfield to Rush Reservoir, 34 468 ft.) was tested again in June and October, 1896, giving $c = 90.04$ as mean of 22 observations. Methods of measurement and range of velocities not stated; "total fall in hydraulic grade line" ranged from 1.08 ft. to 6.95 ft. "Ann. Rep. Exec. Board, Rochester, N. Y.," 1896, p. 45.

15. Experiments by Emil Kuichling on new Rochester conduit between Rush Reservoir and Mt. Hope Reservoir, October and November, 1895. Construction same as No. 14; coated with Sabin's japan. The tabulated results are the mean of three gaugings agreeing closely among themselves. "Ann. Rep. Exec. Board, Rochester, N. Y.," 1894–95, p. 349.

16. Same experiment as Nos. 10 and 11; another portion of conduit; construction the same. Length, 50 965 feet.

17. Experiments by Clemens Herschel on Kearney Extension of East Jersey Water Company. Taper joints, coating "unusually smooth." Conduit put in use January 10th, 1896; experiments made January 21st–30th, 1896 (except one November 18th, 1896). Length, 5 574 ft. Greatest velocity recorded, 4.26. Discharge measured by Venturi meter: loss of head by Bourdon gauges. "115 Experiments," pp. 28, 53.

18. Experiments by Clemens Herschel on conduit No. 2 of East Jersey Water Company, below Pompton Notch. Conduit put in use September 30th, 1896; experiments made in September and October, 1896. Length from 49 833 to 81 139 ft. Highest recorded velocity, 5.41. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 29, 53.

19. Experiments by Clemens Herschel on Conduit No. 1 of East Jersey Water Company, 1892. Observations made on various portions of whole line, lengths ranging from 24 630 to 74 396 ft. Cylinder joints, asphalt coating. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 26, 27, 52.

19*a*. Experiments by Clemens Herschel in 1896. Same conduit as No. 19, but experiments limited to section above Pompton Notch. Length ranging from 10 507 to 24 630 ft. Highest recorded velocity, 4.63. Discharge and loss of head measured as in No. 19. "115 Experiments," pp. 27, 52.

19*b*. Experiments by Clemens Herschel in 1896. Same conduit as No. 19, but experiments limited to portion below Pompton Notch.

Length ranging from 26 610 to 83 000 ft. Highest velocity recorded, 6.06. "115 Experiments," pp. 27, 52.

20. Experiments by Clemens Herschel in 1896 on Conduit No. 2 of East Jersey Water Company, above Pompton Notch. Taper joints; length, 24 648 ft. Velocity ranged from 2.99 to 4.69. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 28, 53.

21. Experiments on conduit of the Pioneer Electric Power Company, Ogden, Utah, August, 1897. Butt joint, asphalt coating. Length, 4 367 ft. and 4 427 ft. Velocity ranged from 0.50 to 3.85. Discharge measured by Venturi meters; loss of head by mercury gauges.

22. Experiments by Clemens Herschel in 1887, on feeding trunk of Holyoke testing flume. Diameter nominally 9 ft.; actual diameter of mean section, 8.58 ft. Cylinder joints; paint coating worn off; rather rusty. Length, 152 88 ft. Discharge measured by "accurate weir"; loss of head by water piezometers (surfaces of water in still boxes determined by hook gauges). *Trans. Am. Soc. C. E.*, pp. 246, 247. "115 Experiments," pp. 29, 54.

It will be noticed that in the case of the authors' results (column 21, Table No. 4), the values of c are given only to the nearest unit, while in every other case the first decimal figure is given. It is not to be inferred from this that these results are regarded as less precise in character than the others. In the opinion of the authors no experiments have been made of such a character as to warrant the use of more than three significant figures in the values of c. In certain cases experimenters have recorded values of c to the sixth significant figure. Such an appearance of precision would be warranted if the results were reliable to within one-thousandth of 1 per cent.

Very little in the way of general conclusions as to the carrying capacity of large riveted conduits can be deduced from the data above recorded. It is evident that the capacity of such conduits is less than was formerly supposed. The experiments of Hamilton Smith on pipes of the smaller sizes (Nos. 1, 3, 4, 6 and 9, Table No. 4) had been supposed to warrant the conclusion that riveted pipes had nearly or quite as great carrying capacity as smooth cast-iron pipes. For such pipes it has been supposed that the value of c increases with the velocity and also with the diameter; the table and curves given by Hamilton Smith*

* "Hydraulics," p. 271 and Pl. XIV. From an inspection of Pl. XIV. it appears that the curves showing the relation between c and v for diameters greater than 1 ft. are based on very meager data, and that the curves as drawn do not conform to the few experimental points recorded. It seems to be arbitrarily assumed that the law of variation of c for large diameters must show a general agreement with the law determined by experiment for small diameters.

being regarded as safe guides in the selection of coefficients. The results recorded in Table No. 4 under numbers 13 to 22 indicate that for large riveted pipes Smith's values are much too great. It is also to be noticed that these results show little or no variation of c with the diameter, and no decided variation with the velocity. It would appear that, for conduits of 36 ins. diameter or over, it is unsafe to count on a value of c greater than 110 for new pipes; if there are frequent or sharp bends the coefficient will doubtless be still less. Moreover, in case of long conduits under ordinary conditions of use, it is to be expected that the capacity will suffer considerable diminution with age.

It should be said that at the time the Ogden conduit was designed, the views of the engineering profession as to the carrying capacity of large riveted conduits fully warranted the use of a coefficient of 120. The construction of the conduit was such that its "smoothness" would certainly appear to compare favorably with any previously made. The butt joints give an interior surface without breaks, there being no interior butt straps at the round joints, while the longitudinal straps are continuous throughout the length of the conduit. As regards smoothness, this construction would appear to have a distinct advantage over either "cylinder" or "taper" joints.

A remark may be made regarding the applicability of Kutter's formula. From Table No. 4 it appears that, for diameters less than 36 ins., and for pipe which is either new or in such condition as to be probably as smooth as when new, the value of n shows little variation in the different experiments, being 0.010 or 0.011 in every case. For larger sizes there is an increase of n with the diameter. This does not appear to be accounted for by a greater roughness of the larger pipes experimented on, but is rather to be attributed to the imperfection of the Kutter formula. It is further true that experiments on the same pipe with different velocities (and consequently different values of the slope s) give different values of n.

III.—Experiments on Wooden Pipe.

Description of Pipe Line.

The plan, profile, dimensions and construction of the entire pipe line may be found in the paper, already referred to.* The plan and

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

profile of the portion of the wooden pipe upon which the experiments were made are shown in Fig. 9, *C* being the lower and *D* the upper pressure station. The diameter of the pipe is 72.5 ins.

Record of Observations.

(*1*) *Example of Plotted Results.*—Reference has already been made to Fig. 5, showing graphically the record of the two manometers and of the Venturi meter difference-gauge for a part of the run made on August 16th on the wooden pipe. Four observations of the series made on the wooden pipe were taken at times of falling pressure, giving manometer records similar to that shown in Fig. 4, although the latter record belongs to an experiment on the steel pipe.

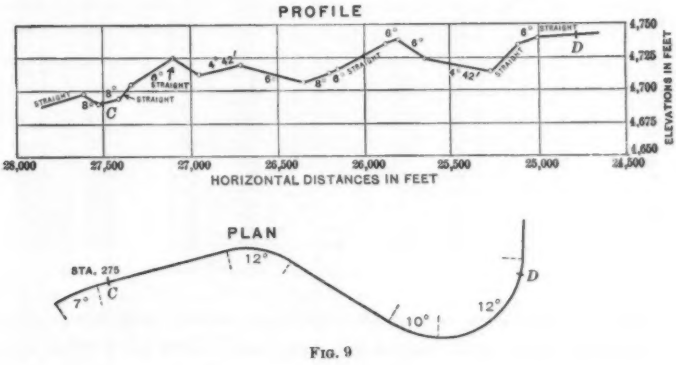

FIG. 9

(*2*) *Tabulation of Reduced Observations.*—From the experiments made upon the wooden pipe, 22 "observations" resulted. The reduced data of these observations are given in Tables Nos. 5, 6 and 7, of which no explanation is needed, since they are in all respects similar to the tables showing data of observations on the steel pipe (Tables Nos. 1, 2 and 3).

The date and period of each observation are given in Table No. 5. It will be noticed that observations 40, 44, 45 and 46 were made under conditions of falling pressure.

(*3*) *General Results.*—Table No. 7 shows the observations arranged in order of the values of the mean velocity of flow, three sets of results

TABLE No. 5.—Wooden Pipe. Discharge Observations.

1	2	3	4	5	6	7	8	9
			North Venturi Meter.		South Venturi Meter.			
No.	Date.	Time.	Reading of difference-gauge. (Feet.)	Rate of discharge. (Cubic feet per second.)	Reading of difference-gauge. (Feet.)	Rate of discharge. (Cubic feet per second.)	Rate of discharge of main pipe. (Cubic feet per second.)	Mean velocity of flow in main pipe. (Feet per second.)
	1897.							
30....	Aug. 15.	8.30— 9.00	.....	.....	0.054	23.4	23.4	.816
31....	" 15.	9.28— 9.45	.....	.....	.178	42.5	42.5	1.482
32....	" 15.	10.14—10.30	.....	.....	.376	61.3	61.3	2.138
33....	" 15.	10.50—11.15	.028	16.7	.366	60.4	77.1	2.689
34....	" 15.	11.32—12.00	.150	38.9	.363	60.1	99.0	3.453
35....	" 16.	8.01— 8.30	.....	.....	.024	15.3	15.3	.534
36....	" 16.	8.45— 8.52	.....	.....	.122	35.2	35.2	1.228
37....	" 16.	8.54— 9.15	.....	.....	.125	35.6	35.6	1.242
38....	" 16.	9.50—10.05	.....	.....	.406	63.6	63.6	2.218
39....	" 16.	10.26—10.45	.028	16.7	.400	63.1	79.8	2.783
40....	" 16.	11.00—11.30	.156	39.6	.395	62.7	102.3	3.568
41....	" 17.	8.30— 9.00	.....	.....	.036	19.1	19.1	.666
42....	" 17.	9.28— 9.45	.....	.....	.145	38.2	38.2	1.332
43....	" 17.	10.16—10.30	.....	.....	.321	56.7	56.7	1.978
44....	" 17.	10.48—11.15	.174	42.1	.390	62.4	104.5	3.645
45....	" 17	11.30—11.41	.174	42.1	.383	61.9	104.0	3.627
46....	" 17.	11.50—12.00	.174	42.1	.380	61.6	103.7	3.617
47....	" 18.	8.30— 9.00	.....	.....	.025	15.6	15.6	.544
48....	" 18.	9.15— 9.45	.....	.....	.120	34.9	34.9	1.217
49....	" 18.	10.25—10.30	.....	.....	.288	53.8	53.8	1.876
50....	" 18.	10.56—11.15	.032	18.1	.297	54.6	72.7	2.536
51....	" 18.	11.36—12.00	.130	36.2	.292	54.1	90.3	3.149

being given as in the case of the steel pipe already discussed. The quantities in this table (values of c, f and loss of head per 1 000 ft.) are also represented graphically in Figs. 10, 11 and 12.

These results differ from those found for the steel pipe in showing a well-marked variation of c and f with the velocity of flow.

Fig. 11 shows that c increases with the velocity in somewhat the way usually assumed for smooth cast-iron pipes. The highest velocity obtained in the experiments was about 3.64 ft. per second. The highest velocity, not accompanied by a fall of pressure, was about 3.45 ft. per second (observation 34). The four observations giving highest velocities are somewhat discrepant, due, probably, to the difficulty of determining accurately the difference between the heights of manometer columns during periods of decreasing pressure.

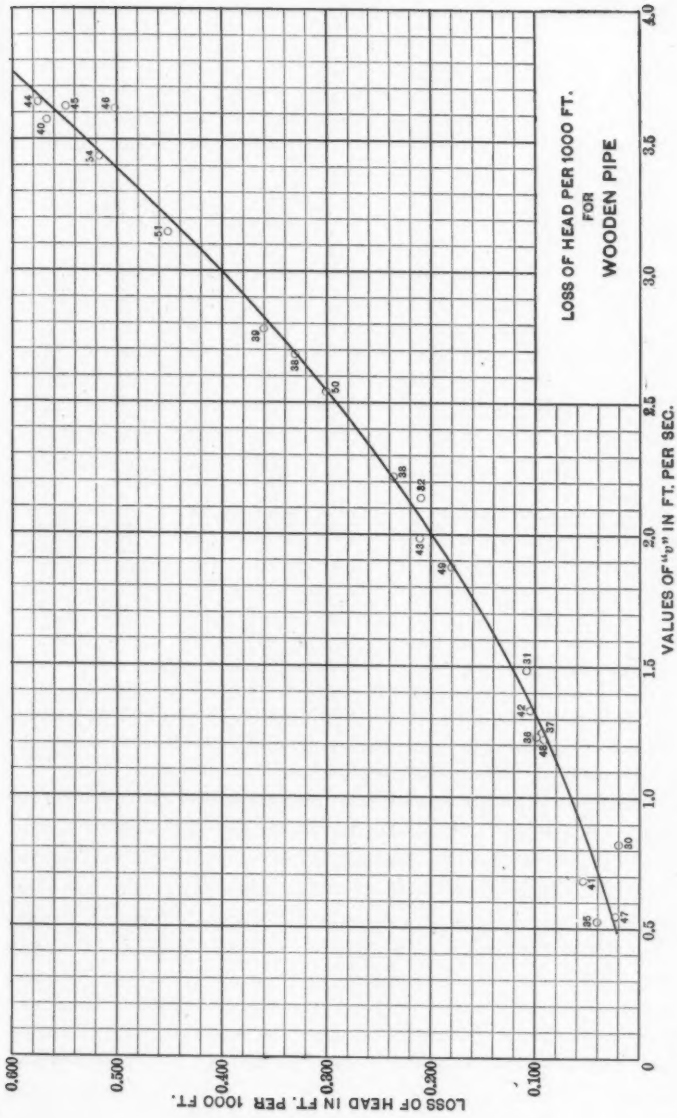
LOSS OF HEAD PER 1000 FT.
FOR
WOODEN PIPE
LOSS OF HEAD IN FT. PER 1000 FT.
0.600
0.500
0.400
0.300
0.200
0.100
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.

FIG. 10.

TABLE No. 6.—Wooden Pipe.—Manometer Observations.

1	2	3	4	5	6	7	8	9
No.	Length of pipe. (Feet.)	Heights of Manometer Columns Reduced to 0° Cent. (Feet.)			Static difference of mercury columns reduced to 0° Cent. (Feet.)	Loss of Head between Manometers.		Loss of head per 1 000 ft. in feet of water.
		Upper.	Lower.	Difference.		Mercury. (Feet.)	Water. (Feet.)	
30..	2 710	3.474	7.364	3.890	3.894	.004	.054	.0201
31..	2 710	3.384	7.256	3.872	3.894	.022	.299	.111
32..	2 710	3.151	7.003	3.852	3.894	.042	.571	.211
33..	2 710	2.882	6.710	3.828	3.894	.066	.898	.331
34..	2 710	2.434	6.225	3.791	3.894	.103	1.401	.517
35..	2 710	3.496	7.382	3.886	3.894	.008	.109	.0401
36..	2 710	3.446	7.320	3.874	3.894	.020	.272	.1003
37..	2 710	3.443	7.318	3.875	3.894	.019	.258	.0952
38..	2 710	3.116	6.963	3.847	3.894	.047	.639	.235
39..	2 710	2.833	6.655	3.822	3.894	.072	.979	.361
40..	2 710	Dropping	Dropping	3.781	3.894	.113	1.537	.566
41..	2 710	3.486	7.369	3.883	3.894	.011	.150	.0552
42..	2 710	3.417	7.290	3.873	3.894	.021	.286	.106
43..	2 710	3.211	7.063	3.852	3.894	.042	.571	.211
44..	2 710	Dropping	Dropping	3.779	3.894	.115	1.564	.576
45..	2 710	"	"	3.785	3.894	.109	1.482	.547
46..	2 710	"	"	3.794	3.894	.100	1.360	.501
47..	2 710	3.492	7.381	3.889	3.894	.005	.068	.0251
48..	2 710	3.443	7.318	3.875	3.894	.019	.258	.0953
49..	2 710	3.252	7.110	3.858	3.894	.036	.490	.181
50..	2 710	2.985	6.819	3.834	3.894	.060	.816	.300
51..	2 710	2.628	6.432	3.804	3.894	.090	1.224	.452
	....	3.518	7.412	3.894	3.894	.000	.000	.000

Reliability of Results.

Excluding observation 30, the experiments on the wooden pipe are more accordant than those on the steel pipe. This is, doubtless, due to the fact that the most important source of error in the steel pipe experiments was absent in the later series.

The greatest pressure to which either manometer was subjected in this series was due to a head of about 101 ft., as against 462 ft. in the former series. Under this lower pressure no difficulty was experienced in securing tight joints, and leakage of mercury did not occur. The long connecting pipe between the manometer and the main pipe (which was necessary at the lower end of the steel pipe) was also avoided, thus diminishing the difficulty of eliminating air. The temperature conditions were, on the whole, as favorable in the second series as in the first; for although neither manometer was furnished with a water jacket, the upper one was very completely sheltered from temperature changes, while the lower was not seriously exposed. The fact that

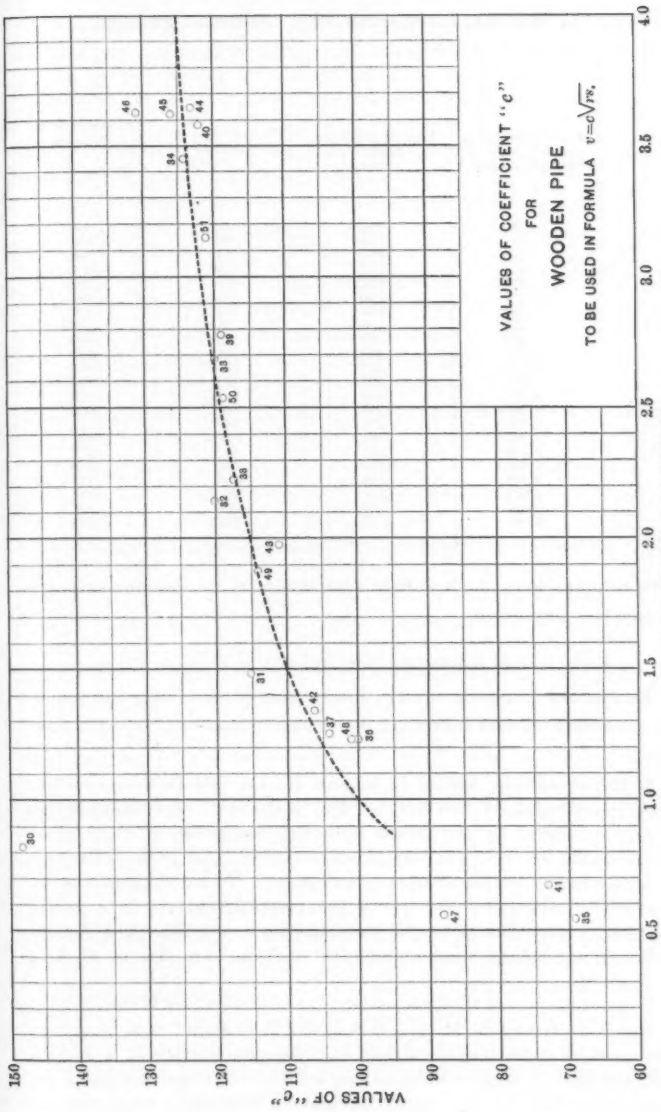
VALUES OF COEFFICIENT "c"
FOR
WOODEN PIPE
TO BE USED IN FORMULA $v=c\sqrt{rs}$.
VALUES OF "c"
150
140
130
120
110
100
90
80
70
60
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.

FIG. 11.

TABLE No. 7.—Wooden Pipe. General Results.

1	2	3	4	5
No.	Mean velocity in main pipe. (Feet per second.)	Loss of head per 1 000 ft. (Feet.)	Value of c in formula $v = c\sqrt{rs}$	Value of f in formula $H' = f\frac{l}{d}\frac{v^2}{2g}$
35.....	.534	.0401	69	.0547
47.....	.544	.0251	88	.0262
41.....	.666	.0552	73	.0483
30.....	.816	.0201	148	.0117
48.....	1.217	.0953	101	.0250
36.....	1.228	.1008	100	.0259
37.....	1.242	.0952	104	.0241
42.....	1.332	.106	106	.0230
31.....	1.482	.111	115	.0195
49.....	1.876	.181	114	.0199
43.....	1.978	.211	111	.0209
32.....	2.138	.211	120	.0179
38.....	2.218	.235	117	.0186
50.....	2.536	.300	119	.0182
33.....	2.689	.331	120	.0178
39.....	2.783	.361	119	.0181
51.....	3.149	.452	121	.0177
34.....	3.453	.517	124	.0168
40.....	3.568	.566	122	.0173
46.....	3.617	.501	131	.0149
45.....	3.627	.547	126	.0161
44.....	3.645	.576	123	.0168

both columns were much shorter also rendered the temperature correction less important.

Known Data Regarding the Capacity of Wooden Pipes.

Besides the foregoing results, the authors know of only the following experimental data regarding the carrying capacity of wooden stave pipe.

1. Experiment by Arthur L. Adams, M. Am. Soc. C. E., on conduit of Astoria, Oregon, Water-Works. This is the same experiment as that already described in connection with the discussion of steel pipes (No. 5, Table No. 4). The loss of head in 4 188 ft. of 18-in. stave pipe was measured by means of open stand-pipes. The discharge was determined by measuring the rise of the reservoir surface in a test of eighteen hours. The value of c was 132.9 for $v = 3.605$. The Kutter coefficient of roughness was about 0.010.—*Trans. Am. Soc. C. E.*, Vol. xxxvi, p. 26.

2. In an account of the 30-in. stave pipe of the Denver Water-Works, by James D. Schuyler, M. Am. Soc. C. E., it is stated that gaugings were made by measuring the rise of the reservoir surface in a known time, and also by measurement of the velocity by current meter. As a result it is said that in "applying Kutter's formula to wood pipe, as

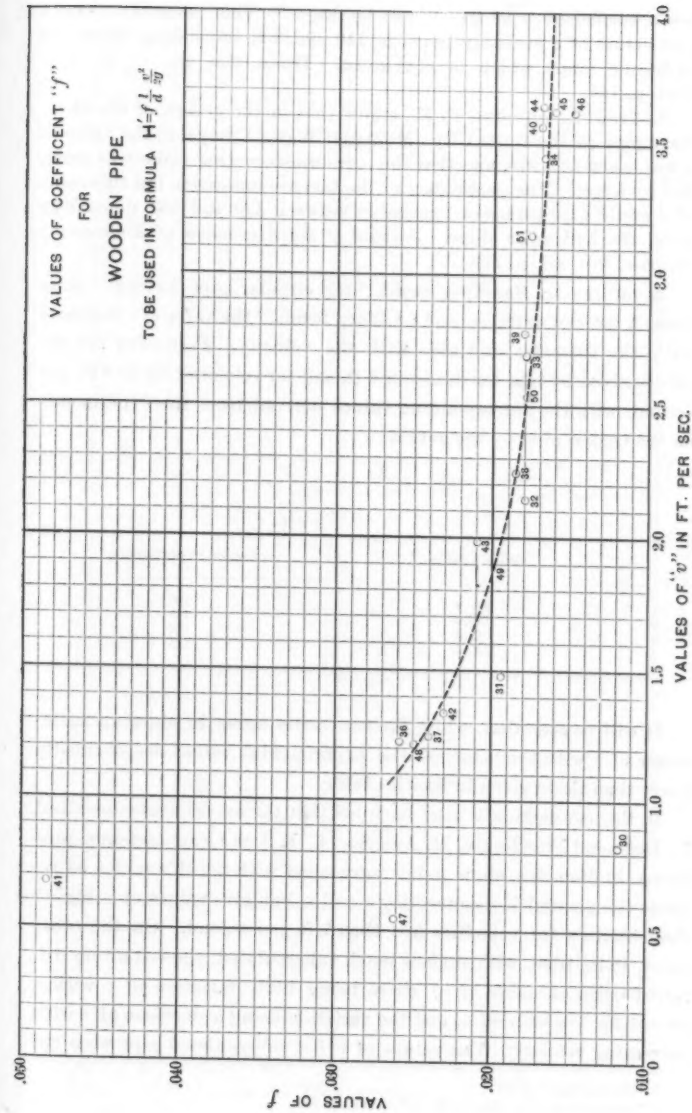
VALUES OF COEFFICENT "f"
FOR
WOODEN PIPE
TO BE USED IN FORMULA $H=f\frac{l}{d}\frac{v^2}{2g}$
VALUES OF f
.050
.040
.030
.020
.010
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.

FIG. 12.

low a coefficient n as 0.0096 can be used." This value of n would give values of c probably between 140 and 150, depending upon the hydraulic slope, which is not given. *Trans. Am. Soc. C. E.*, Vol. xxxi, p. 144.

3. Fred. B. Gutelius, C. E., states that in the design of the 24-in. stave pipe of the Butte City (Montana) Water Company, the value of n was taken as 0.010, and that the correctness of this value was justified by a test. The particulars of the test are not given, but this value of n would give values of c probably between 120 and 130, depending upon the hydraulic slope. *Journal of the Association of Engineering Societies*, Vol. xii, p. 219.

According to Hamilton Smith,* for circular pipe having "quite smooth interior surfaces, and no sharp bends," the value of c increases with the diameter and also with the velocity. Following are the values given by him for diameter 6 ft. and for velocities up to 4 ft. per second, with the corresponding values derived from the experiments on the Ogden pipe by the authors:

v	c	
	Smith's table.	Ogden experiments.
1	131.8	100
2	138.0	115
3	142.3	122
4	145.5	125

It will be seen that, while the two series agree in showing an increase of c with the velocity, the experimental values are decidedly lower than those given in Smith's table.

In this connection it may be noted that the experiments described by Desmond FitzGerald, M. Am. Soc. C. E.,† on clean cast-iron pipe 48 ins. in diameter, show a fair agreement with Smith's tables as regards the general law connecting v and c, but give values of c higher than Smith's for velocities less than 7 ft. per second. On the other hand, 48-in. pipe, with surface much tuberculated, showed a very different series of values of c; there being little variation of c with v except for low velocities, and the variation being a decrease of c with increasing velocity. The values of c for tuberculated pipe were not

* "Hydraulics," p. 271.

† *Transactions*, Am. Soc. C. E., vol. xxxv., p. 241.

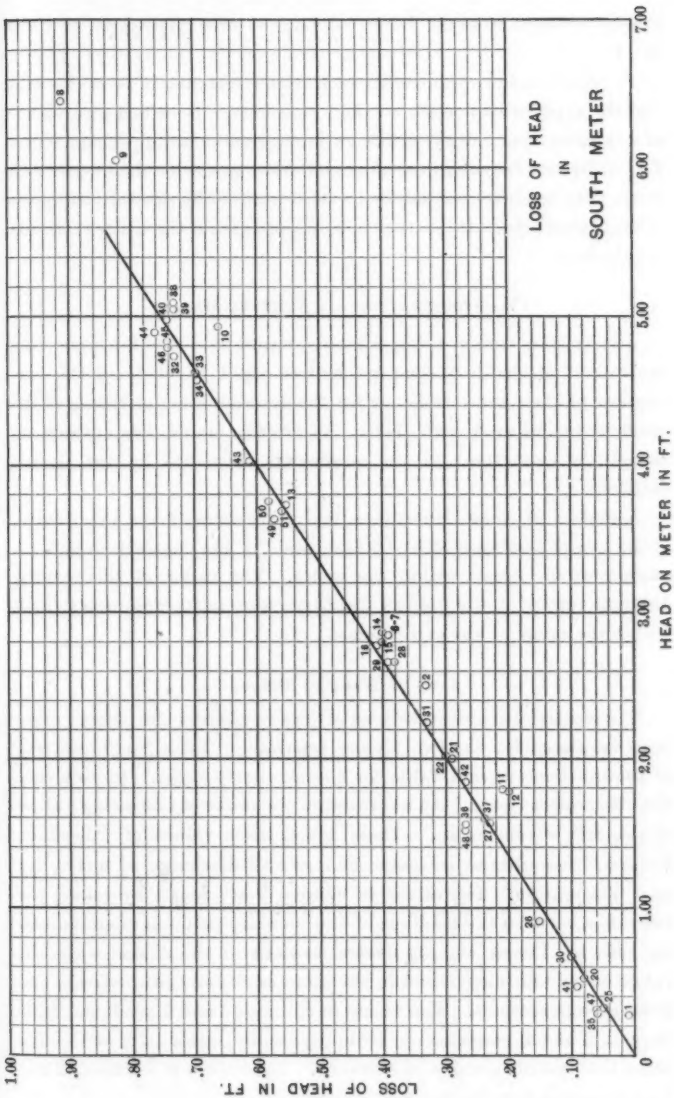
LOSS OF HEAD IN SOUTH METER
LOSS OF HEAD IN FT.
HEAD ON METER IN FT.
0
1.00
2.00
3.00
4.00
5.00
6.00
7.00
.10
.20
.30
.40
.50
.60
.70
.80
.90
1.00

FIG. 18.

greatly different from the values for large riveted pipe given in Table No. 4.

In regard to the applicability of Kutter's formula it is to be said that the experiments on the wooden pipe herein described give values of n ranging from 0.012 to 0.015, an average value being perhaps 0.013. The difference between this value and those given for the Denver and Butte City conduits can hardly be attributed to the greater roughness of the Ogden pipe. It is rather to be supposed that the Kutter formula is defective.

IV.—Observations on Venturi Meters.

The Venturi meters being the only means available for measuring the rate of discharge of the pipe, no test could be made of the correctness of their indications, or for the determination of values of the coefficient to be used with them. As already stated, the authors assumed the correctness of the coefficients furnished by the manufacturers.

A matter of some importance, both theoretically and practically, is the loss of head caused by friction, and by the contraction and expansion of the stream within the meter. The values of this loss for different values of the rate of discharge were fairly well determined within the range of the experiments.

Loss of Head in Meters.

The results of the observations of loss of head in the meters have been tabulated for the two meters separately, Table No. 8 referring to the south meter, and Table No. 9 to the north meter. In each case the observations have been arranged in the order of increasing values of the rate of discharge. These tables show values of "head on Venturi," loss of head in meter, and rate of discharge of meter, for each observation. The values in column 2 are found by taking the difference between the readings of the mercury columns communicating with the throat and up-stream sections of the meter; while the values in column 4 are found in the same way from the up-stream and down-stream sections. The values in columns 3 and 5 are found from those in 2 and 4, respectively, by applying the factor 12.6, or $e - 1$, e being the specific gravity of mercury. The values in columns 2 and 6 are repeated from Tables Nos. 1 and 5.

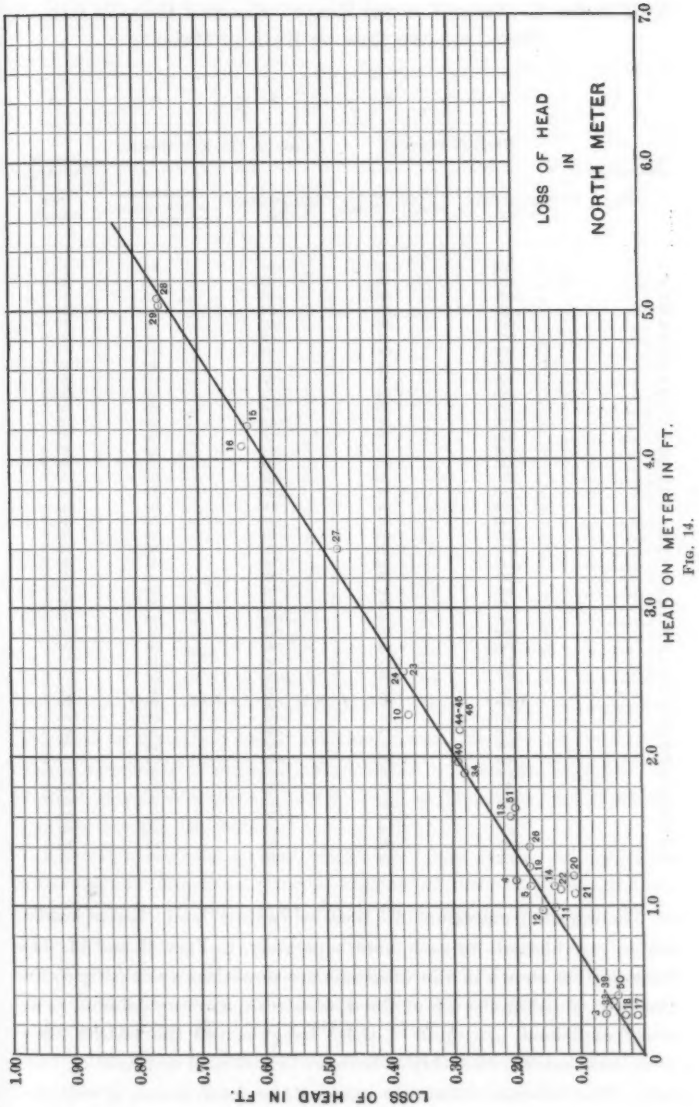
LOSS OF HEAD
IN
NORTH METER
LOSS OF HEAD IN FT.
HEAD ON METER IN FT.
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00

FIG. 14.

TABLE No. 8.—SOUTH VENTURI METER. RELATION BETWEEN RATE OF DISCHARGE AND LOSS OF HEAD IN METER.

1	2	3	4	5	6
Observation No.	HEAD ON VENTURI.		LOSS OF HEAD IN METER.		Rate of discharge. (Cubic feet per second.)
	Gauge reading.	Equivalent water column. (Feet.)	Gauge reading.	Equivalent water column. (Feet.)	
1	.022	.28	.001	.01	14.5
35	.024	.30	.005	.06	15.3
47	.025	.32	.005	.06	15.6
25	.026	.33	.004	.05	16.0
41	.036	.46	.007	.09	19.1
20	.043	.54	.006	.08	21.0
30	.054	.68	.008	.10	23.4
26	.072	.91	.012	.15	27.2
48	.120	1.51	.021	.27	34.9
36	.122	1.54	.021	.27	35.2
27	.123	1.55	.018	.23	35.3
37	.125	1.58	.019	.24	35.6
12	.140	1.76	.016	.20	37.5
11	.141	1.78	.017	.22	37.6
42	.145	1.83	.021	.27	38.2
21	.159	2.00	.023	.29	40.0
22	.160	2.02	.024	.30	40.1
31	.178	2.24	.026	.33	42.5
2	.199	2.51	.026	.33	44.8
29	.210	2.65	.031	.39	46.1
28	.211	2.66	.030	.38	46.2
16	.219	2.76	.033	.41	46.9
15	.221	2.79	.032	.40	47.1
6	.225	2.84	.031	.39	47.5
7	.225	2.84	.031	.39	47.5
14	.226	2.85	.032	.40	47.5
49	.288	3.63	.045	.57	53.8
51	.292	3.68	.044	.56	54.1
13	.296	3.73	.042	.53	54.5
50	.297	3.74	.046	.58	54.6
43	.321	4.04	.048	.61	56.7
34	.363	4.57	.055	.69	60.1
33	.366	4.61	.055	.69	60.4
32	.376	4.74	.058	.73	61.3
46	.380	4.79	.059	.74	61.6
45	.383	4.83	.059	.74	61.9
44	.390	4.91	.060	.76	62.4
10	.391	4.93	.052	.66	62.5
40	.395	4.98	.059	.74	62.7
39	.400	5.04	.058	.73	63.1
38	.406	5.12	.058	.73	63.6
9	.481	6.06	.065	.82	69.2
8	.512	6.45	.072	.91	71.2

The simultaneous values of "head on Venturi" and "loss of head in meter" are plotted, for each meter separately, in Figs. 13 and 14. The irregularities shown in the diagrams are doubtless due chiefly to the presence of dirt at the top of the down-stream mercury column, as already explained. In spite of these irregularities the results show quite satisfactorily the relation between loss of head and head on Venturi. This relation, within the range of the observations, is well rep-

TABLE No. 9.—North Venturi Meter. Relation Between Rate of Discharge and Loss of Head in Meter.

1	2	3	4	5	6
Observation No.	Head on Venturi.		Loss of Head in Meter.		Rate of discharge. (Cubic feet per second.)
	Gauge reading.	Equivalent water column. (Feet.)	Gauge reading.	Equivalent water column. (Feet.)	
17	.021	.26	.001	.01	14.1
18	.021	.26	.002	.03	14.1
3	.022	.28	.005	.06	14.5
33	.028	.35	.004	.05	16.7
39	.028	.35	.004	.05	16.7
50	.032	.40	.008	.04	18.1
12	.077	.97	.013	.16	28.1
11	.078	.98	.011	.14	28.2
21	.086	1.08	.009	.11	29.6
22	.087	1.10	.010	.13	29.8
14	.090	1.13	.011	.14	30.3
5	.090	1.13	.014	.18	30.3
4	.093	1.17	.016	.20	30.9
20	.095	1.20	.009	.11	31.2
19	.099	1.25	.014	.18	31.7
26	.111	1.40	.014	.18	33.7
13	.127	1.60	.017	.21	35.8
51	.130	1.64	.016	.20	36.2
34	.150	1.89	.022	.28	38.9
40	.156	1.97	.023	.29	39.6
44	.174	2.19	.023	.29	42.1
45	.174	2.19	.023	.29	42.1
46	.174	2.19	.023	.29	42.1
10	.181	2.28	.029	.37	42.8
23	.204	2.57	.030	.38	45.5
24	.205	2.58	.030	.38	45.7
27	.270	3.40	.038	.48	51.8
16	.323	4.07	.050	.63	57.0
15	.333	4.21	.049	.62	57.8
29	.400	5.04	.060	.76	63.1
28	.401	5.06	.060	.76	63.2

resented by a straight line. In other words, the loss of head in the meter appears to be directly proportional to head on Venturi. The loss of head thus appears to vary nearly as the square of the velocity of flow through the meter.

If H denotes head on Venturi and H'' loss of head in meter, the above relation is expressed by the equation $H'' = a\,H$, where a is a constant. The value $a = 0.149$ for each meter agrees well with the experimental data.

The relation between loss of head in meter and rate of discharge, for the Ogden meters, using the relation $H'' = 0.149H$, is shown by the curve in Fig. 15.

The experience of the authors in these tests has convinced them of the great value of the Venturi meter in experimental work, requiring

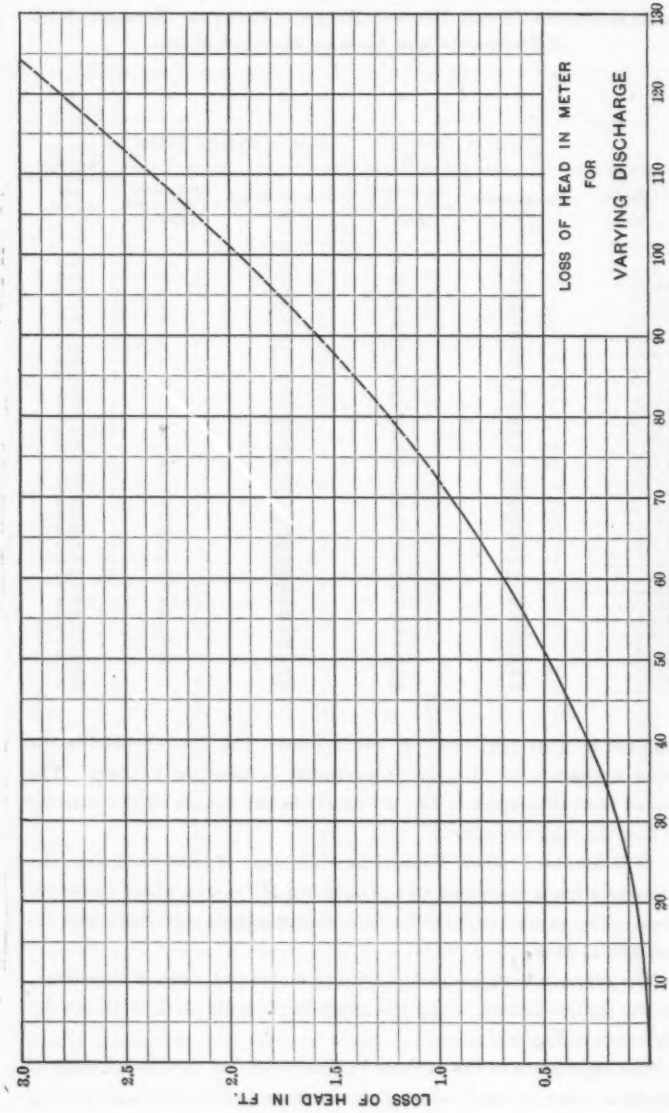
LOSS OF HEAD IN METER
FOR
VARYING DISCHARGE
LOSS OF HEAD IN FT.
3.0
2.5
2.0
1.5
1.0
0.5
10
20
30
40
50
60
70
80
90
100
110
120
130
DISCHARGE IN CU. FT. PER SEC.

FIG. 15.

the measurement of the rate of discharge of pipes. By no other method would it have been possible to collect so much information, at all comparable in accuracy with that obtained in these experiments, without months of labor. Moreover, the sensitiveness of the difference-gauge to slight changes of the rate of discharge shows that the meter is capable of giving measurements of great precision. It is perhaps not too much to say, that, except for very small quantities, the Venturi meter furnishes the most precise of all methods of measuring the rate of discharge, provided the values of the coefficient are accurately known.*

In carrying out these experiments the authors were assisted by Mr. L. S. Boggs, E. E., Superintendent of the Pioneer Electric Power Company; by Mr. F. W. Hart, Assistant Superintendent, and by Mr. L. B. Spencer, student in civil engineering in Stanford University. To them, for the valuable and efficient aid rendered; to the directors of the Pioneer Electric Power Company, and especially to Chief Engineer Bannister, for all courtesies rendered, the authors desire to express their sincere thanks.

* In regard to the range of the coefficients of a Venturi meter, the only experimental results known to the authors are those of Herschel. These results, obtained with meters of diameters of 12 ins., 48 ins. and 108 ins., indicate that the values of the coefficient, for throat velocities between 5 ft. and 25 ft. per second, are not likely to differ from unity by more than 3% (see "115 Experiments," Pl. III, p. 43). Even with an unrated meter it is probable that the results obtained will compare favorably in accuracy with those obtainable by any other method of measurement.

APPENDIX.

ATTACHMENT OF PIEZOMETERS.

Object of discussion.

It is proposed to consider the question whether the height of a piezometer column depends upon the position of the point of attachment to the pipe; and also whether different results will be given by multiple attachment than by attachment at a single point. The discussion will refer to a vertical cross-section of a horizontal pipe; but the reasoning will be seen to apply to the transverse section of a pipe having any direction.

Variation of Pressure in a Cross-Section.

The pressures at different points in the same cross-section are unequal, being less near the top of the pipe than near the bottom. If the particles of water all moved in straight lines parallel to the axis of the pipe, the law of variation of pressure with depth would be the same as for water at rest. Any variation from this law must be due to the fact that particles are deflected from the straight axial direction; in other words, to the fact that some of the particles of water passing a given section at any instant are being accelerated vertically. If such is the case, the distribution of the vertical components of acceleration throughout the cross-section will continually vary; so that not only will the law of variation of pressure with depth differ from the hydrostatic law, but the actual pressure at any given point will vary. Therefore, in any case in which a piezometer is found to give results which fluctuate but slightly, it may be concluded that the sinuosities in the motions of the particles of water have no important effect upon the law of variation of pressure in the cross-section. Experience bears out the supposition that this condition is satisfied when steady flow has been maintained for some time in a straight pipe of uniform cross-section. The following discussion will therefore proceed on the assumption that the pressure in any cross-section varies with the depth, according to the hydrostatic law.

Case of Open-Topped Piezometer.

If the pressure throughout a cross-section exceeds that of the atmosphere, and if open tubes are attached at two points, as at A and B (Fig. 16), it is evident that water will rise to the same level in both. For since the pressure at B exceeds that at A by an amount equiva-

lent to the height (h) of A above B, the corresponding pressure columns must differ in height by the same amount h. In fact, the pressure at any two points C and D at the same level in the two pipes must be equal.

If the pressure at the top of either column is less than atmospheric, that column will stand correspondingly higher; but the surfaces X and Y will be at the same level so long as they are subject to equal pressures.

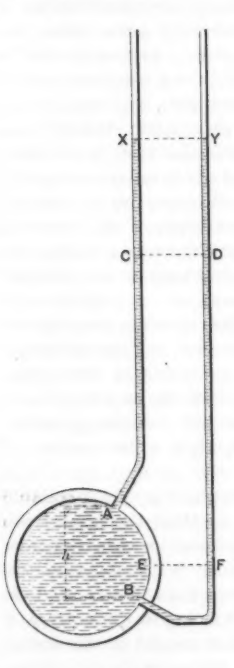

FIG. 16.

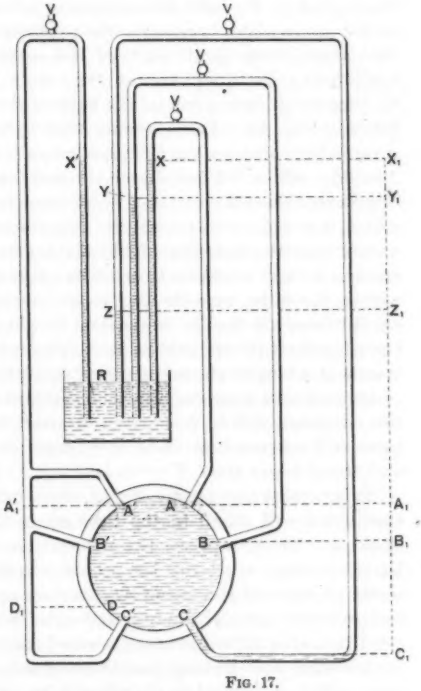

FIG. 17.

Case of Vacuum Gauge.

In case the pressure throughout the cross-section of the pipe becomes less than one atmosphere, the open piezometer becomes inapplicable. It is, however, instructive to consider what will happen if, with open tubes attached as in Fig. 16, the pressure within the pipe gradually diminishes. The surfaces X and Y will fall equally until the pressure within the pipe at the level A becomes just one atmosphere, the surface Y then standing on a level with A. The press-

ure at any point lower than A is still greater than one atmosphere. Let the pressure in the pipe diminish still further until there is just one atmosphere of pressure at some level E, between A and B; pressures less than one atmosphere existing at all points higher than E, and greater than one atmosphere at all points lower. The surface Y will fall to F, level with E; while at A air will be drawn into the pipe. If the level of atmospheric pressure falls below B, air will also enter at B.

Suppose, next, that the tubes attached at A and B are arranged as shown in Fig. 17, both tubes dipping into a reservoir whose surface is under atmospheric pressure, the pressure within the pipe being less than atmospheric in all parts of the cross-section. At first let air be admitted to the tubes through the valves V V, which are then closed. So long as air enters freely, the tubes act in the same way as the open tubes of Fig. 16. The pressure within the piezometer tubes being greater than that within the pipe, air enters the pipe from both tubes. After the valves V V are closed, the withdrawal of air causes a decrease of pressure in each tube, and water rises from the reservoir to a height which, for each tube, represents the difference between the pressure within the tube and that of the atmosphere at the surface of the reservoir. Air will continue to enter the pipe at A so long as the pressure within the tube exceeds the water pressure at A. A condition of equilibrium will finally be reached in which the pressure throughout the air in the tube is equal to the water pressure at A, and the surface X stands at a height above the reservoir surface representing the excess of atmospheric pressure above the pressure within the space A X. A like condition will be reached in the other tube, but since the pressure-head at B exceeds that at A by an amount $A_1 B_1 = h$, the surface Y will stand lower than X by an amount $X_1 Y_1 = h$.

It is further to be noticed that, if the opening at A is of appreciable size, water will stand in the tube up to the level of the top of the opening. Thus, suppose the exhaustion of the air within the tube has proceeded until the air pressure in the tube is greater than the water pressure at the top of the opening and less than that at the bottom; air will enter at the top, and water will flow out at the bottom, a condition of equilibrium being reached only when no part of the stream in the main pipe flowing past the orifice is in contact with air. Similarly, water will stand in the other tube up to the level of the top of the opening B.

It thus appears that it makes no difference at what point of the cross-section the piezometer is attached, provided the indicated pressure be referred to the proper level. If the apparatus is attached and operated in the manner above described, the piezometer reading in each case gives the pressure within the pipe at points which are on a level with the top of the orifice leading from the pipe to the piezometer tube.

If, after the condition of equilibrium above described has been reached, the pressure within the pipe increases, but still remains less than that of the atmosphere, water will rise in the tubes at A and at B, the surfaces X and Y at the same time falling. A condition of equilibrium will ensue in which the height of the column RX exceeds that of the column above A by an amount equivalent to the difference between atmospheric pressure and the pressure at A, with a similar condition of affairs in the tube attached at B. In such a case both columns in the same piezometer tube must be observed in order that the pressure at a given level in the main pipe may be known.

A further case to be considered is that in which some part of the piezometer tube is lower than the opening into the pipe, as at C, Fig. 17, the pressure within the pipe at C being supposed less than one atmosphere. If the tube is at first filled with air under atmospheric pressure, air will enter the pipe at C until the pressure within the tube falls to the value of the water pressure at the bottom of the opening. As soon as the air pressure becomes less than this value, water will flow into the tube through the lower part of the orifice, collecting in the lowest part of the tube until its cross-section is entirely filled. Water will continue to flow into the tube at the bottom of the orifice, and air to enter the pipe at the top, until no air remains in contact with the water flowing past the orifice in the main pipe. A condition of equilibrium will finally be reached in which water fills the tube from C to some point C_1. This point C_1 will be high enough so that the lowest cross-section of the tube is completely filled, but its exact height cannot be predicted. The height of the column RZ' will measure the amount by which atmospheric pressure exceeds the pressure within the pipe at the level C_1.

Consider next the case of a single piezometer communicating with the given cross-section of the pipe at two points, as at A' and C', Fig. 17, the pressure at every point of the cross-section being less than that of the atmosphere. As before, let the valve V be first open and then closed. While it is open, air will enter the pipe at A' and at C'. After it is closed air will continue to enter the pipe so long as the pressure within the tube exceeds the water pressure, either at A' or at C'. At the same time water will rise from the reservoir R to a height measuring the amount by which the pressure within the tube is less than one atmosphere. When the pressure within the tube becomes less than that in the pipe in the lower portion of the opening C', air will cease to enter at this part of the opening, and water will flow into the tube. At the top of the opening air will still enter until the air pressure falls below the water pressure at that point; after which water will flow out through the entire opening. Air will continue to enter the pipe at A', and the air pressure in the tube to decrease. When the pressure in the tube is equal to the water pressure at some point BD, between A'

and C', water tends to stand in the tube at the height D_1; no condition of equilibrium is reached, however, so long as air continues to enter at A'. Finally the pressure in the tube will reach the value of the water pressure at the highest point of the orifice A', air will cease to enter the main pipe, and water will stand in both branches of the tube to the level $A'\,A'_1$. The air pressure in the tube is now equal to the pressure in the water at the level A', and is less than atmospheric pressure by an amount equivalent to the water column $R\,X'$.

Summary of Results.

The conclusions to be drawn from the foregoing discussion may be summarized as follows:

(1) When the pressure in the given cross-section of the pipe everywhere exceeds that of the atmosphere, an open piezometer will stand at the same height at whatever point of the cross-section it be attached, and whether it communicates with the pipe at one point or at several.

(2) If a "vacuum" piezometer be used, the pressure at the point of attachment being less than that of the atmosphere, the pressure at any level in the pipe can be inferred if the water surfaces in both branches of the piezometer are known. The pressure within the main pipe, at all points on a level with the surface of the column in the branch of the piezometer tube adjacent to the pipe, has "negative" value measured by the height of the column in the other branch above the open reservoir.

(3) In case of a vacuum piezometer connected with the pipe at more than one point in the same cross-section, water will stand at the same level in all the connecting tubes.

(4) In case of a vacuum piezometer, if air be freely admitted to the tube before the observation, the pressure within the main pipe remaining constant, water will rise in the connecting tubes, whether one or several, to the level of the highest point of communication with the pipe.*

The authors have not been able to subject these conclusions to the test of experiment. They are, however, in conformity with the results noticed and discussed by Herschel.† For measuring pressures in the throat of a Venturi meter he used a vacuum piezometer communicating with the throat at several points, the several branch tubes being furnished with valves, so that communication could be opened through any one or more of them at pleasure. From the working of the piezometer with different combinations of tubes open, Herschel concludes that:

"The Venturi must, in all cases, be pierced for connection with the air chamber vertically at its crown, and may be pierced radially at as

* Unless the tube leading to this point runs below the level of the opening into the pipe.

† *Transactions*, Am. Soc. C. E., Vol. xvii, pp. 248, 250, 251.

many additional points as we please, without affecting the reading of the standard crown orifice."

The foregoing theory suggests no reason for the conclusion that the pipe must be pierced at the highest point of the cross-section; but Herschel's conclusion is otherwise in strict conformity with the principles reached above, and summarized in the paragraphs numbered (3) and (4). These principles are not fully tested by the experiments recorded, since in every case bearing upon the point under discussion one of the open passages was the one at the top of the cross-section.

For the case of pressures greater than atmospheric, Herschel's experiments of October, 1887,* furnish several instances in conformity with the conclusion above drawn and stated in paragraph (1). This is, in fact, the conclusion reached by Herschel. In the experiments of the authors at Ogden no pressures less than atmospheric had to be measured.

The foregoing discussion has not referred to the effect of air carried by the water and collecting in the piezometer tube. Attachment of the piezometer at the top of the pipe probably gives greater liability to trouble from this source than attachment at a lower point; but in all cases means must be provided for getting rid of whatever air may collect.

* *Transactions*, Am. Soc. C. E., Vol. xvii, p. 250.

DISCUSSION.

Mr. Williams. GARDNER S. WILLIAMS, Assoc. M. Am. Soc. C. E.—Some information in regard to the calibration of the instruments used in the experiments under discussion, if they were calibrated at all, would seem to be desirable. The speaker has found that there is sometimes a variation in the specific gravity of mercury, and also that some instrumental coefficients may possibly enter into the question, notably in the connection with the lower gauge. The statement that an air-cock is provided at the highest point in this connecting pipe offers an opportunity for suspicion as to the results afforded by that gauge. From some observations which the speaker has made, he has learned that there is nothing more delusive than an air-cock. If it is not placed at the high point, or if the air bubble is not at the highest point and properly blown off, the entire test for that velocity and for that time may be vitiated, and, unless the observers were able to assure themselves that there was no accumulation of particles of air in the connection of their gauge, little dependence can be placed on the results they have given. It seems to the speaker that the lack of coherence of the observations and the apparent sinuosity of the mean line for the value of n, indicate that there was something very materially the matter with the observations. He does not wish to appear unduly critical of those who carried out these experiments, fully appreciating the difficulties under which they labored, yet he does not think these experiments are entitled to rank with the experiments heretofore made and recorded by Messrs. Desmond FitzGerald, John R. Freeman, and others. The same objection might be made to most of the experiments made in the West, on which dependence is placed for knowledge as to the carrying capacity of large conduits. They have been ordinarily single experiments by inexperienced experimenters, who are not aware of the conditions likely to be met, and the results obtained are not entitled to the same credence as the observations of those who have devoted years of study to the subject. On the line under discussion the effect of curvature must have been of some importance. Without wishing to go into that subject very extensively, the speaker desires to state positively that the line of maximum velocity of the water passing around a curve is not in the axis of the pipe. He fully agrees that more extended experiments are highly desirable, and would also suggest that, with shorter lengths and more delicate instruments, possibly more satisfactory results could be obtained.

Mr. Trautwine. JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—Mr. Herschel, in his paper* describing the Venturi meter as he designed it in the first

* *Transactions*, Am. Soc. C. E., Vol. xvii, p. 228, and Vol. xviii, p. 133.

Mr. Trautwine.

place, seems to assume that the elevation of the columns in the piezometers might properly be measured from the top of the pipe; but Professor Merriman* remarks: "It is thought that the elevation should be measured to the center of the section rather than to the top." Prior to the appearance of the present paper, these expressions seem to have comprised all the published matter on the subject. The authors have, therefore, rendered valuable service in entering into an elaborate discussion of this point in the appendix to their paper.

In Philadelphia, where the Venturi meter, in sizes up to 48 ins., has been extensively used, experiments have been made comparing the loss of head through different meters, the Venturi being compared with several of the ordinary mechanical meters. In these experiments the Venturi gave, at a given velocity, much less resistance to flow than any of the others, with the single (and somewhat singular) exception of the "Gem" meter, which, at high velocities, gave even a shade less resistance than the Venturi.

In all, the resistance increased in proportion to the head, or to the square of the velocity.

The range of velocities within which any given Venturi meter will work satisfactorily is rather limited, and it is important, therefore, to determine in advance, to what range of velocities the main in question is to be subjected, and to design the throat ratio of the Venturi accordingly, a high throat ratio (throat small relatively to main) being necessary to secure sufficient difference between the piezometer heights under low velocities, and a low throat ratio (throat wide relatively to pipe) being required to avoid throttling or "wire drawing" under high velocities.

Hence, when the Venturi is placed in a main where the normal velocities are low, and where the main may yet be occasionally drawn upon for abnormally high velocities, as in case of fire, a by-pass should be provided in order that the Venturi may be cut out of service at such times of abnormal demand.

Mr. Goldmark.

HENRY GOLDMARK, M. Am. Soc. C. E.—In the Venturi meter, as used at Ogden, there are four piezometer openings; that is, a complete chamber surrounds the Venturi tube, and this chamber is tapped for the pressure pipe, but communicates with the interior of the pipe, at the top, the bottom and the two sides.

With reference to Mr. Henny's discussion, the speaker is familiar with some matters which, perhaps, deserve mention. One is with reference to the amount of sand that could have been brought into the pipe. The pipe was not really put into use until May, 1897, and it is very doubtful whether there was a continuous flow through it until some time in June. The experiments were made in August, so that the period of time during which there was a chance for sediment

* "A Treatise on Hydraulics," 1889, p. 161.

Mr. Goldmark. to enter was very short. It is true that the blow-off valves would probably not remove such sediment. It is also true that the so-called air-valves were not intended primarily for removing the air which might accumulate at summits. Their construction is such that the air cannot escape automatically, though it might be possible to release it by pressing on the top of the valves with a bar. It would not be easy to do so, and there is nothing in the paper to show that it was done before the experiments were made. The main purpose of these valves was to act as a safeguard in case of a break in the pipe, as they are designed to admit air at a high velocity to take the place of the water flowing off, and thus prevent the collapse of the pipe.

Mr. Kuichling. E. KUICHLING, M. Am. Soc. C. E.—This paper is an extremely valuable addition to hydraulic literature, and the experiments described therein can fairly be ranked with the best that have yet been made with large pipes. The authors appear to have done their work with close attention to the many details that are essential to accuracy, and have left little room for criticism. Their only fault, in the speaker's opinion, consists in not having explained sufficiently the reason for having adopted a uniform value of 13.60 for the specific gravity of the mercury which they used in their manometers. By this omission they may either entrap a reader of the paper into expressing dissatisfaction with this value, or involve him in a tedious examination of the subject, inasmuch as the authors' method of computing the equivalent height of a column of water from the observed height of a column of mercury appears at first sight to be somewhat inaccurate.

It is stated in the paper that the temperature of the water in the pipe ranged from 16° to 18° Cent., and that the observed heights of the mercury columns in the several manometers, at various other temperatures not given, were reduced to equivalent heights at the temperature 0° Cent.; also that these latter heights were then multiplied uniformly by the specific gravity (13.60) of the mercury, in order to obtain the height of the equivalent water columns at the aforesaid temperatures. Now, to those who are aware that the specific gravities of both mercury and water vary with considerable changes in the temperature of these liquids, this manner of obtaining the equivalent height of the water column cannot fail to appear to be of doubtful accuracy. On further investigation, however, such doubt will disappear, particularly when it is remembered that the manometers or mercury columns are not used to measure great differences in elevation, but only the comparatively small losses of head by friction in the large pipe; and it may also be remarked incidentally that the ingenious manner in which the authors have eliminated the constant difference in level between the manometer stations, and left in view only the friction loss of head in the pipe, is worthy of unstinted praise.

When an open mercury column is used to determine, from the

Mr. Kuichling.

water pressure in a pipe, the difference in elevation of the corresponding points of attachment, it becomes necessary to pay close heed to the respective temperatures of the water and mercury, and perhaps also to calibrate the manometer under the particular conditions of temperature prevailing at the time of the observations. As already mentioned, the relative specific gravity of mercury varies with the temperature, and to exhibit this variation within the range of temperatures usually encountered in water-gauging operations, Table No. 10 is submitted. The weights per cubic foot of distilled water at the different temperatures, given in column 3, are taken from the carefully prepared paper on the subject by Mr. A. F. Nagle, M.E., read before the American Society of Mechanical Engineers in May, 1892, and the weight of pure mercury at 0° Cent., has been taken at 848.75 lbs. per cubic foot, as given by all well-recognized authorities:

TABLE No. 10.—EXHIBITING RELATIVE SPECIFIC GRAVITIES AND WEIGHTS PER CUBIC FOOT OF MERCURY AT DIFFERENT TEMPERATURES; ALSO WEIGHTS PER CUBIC FOOT OF DISTILLED WATER AT DIFFERENT TEMPERATURES.

1	2	3	4	5	6	7	8	9
Temperature.		Weight of 1 cu. ft. of water. Lbs.	Weight of 1 cu. ft. of Mercury, by Regnault's formula. Lbs.	Relative specific gravity of Mercury.	Weight of 1 cu. ft. of Mercury, as per approximate rule of authors. Lbs.	Relative specific gravity of Mercury.	Weight of 1 cu. ft. of Mercury at 0° Cent. Lbs.	Relative specific gravity of Mercury.
Cent.	Fahr.							
0°	32°	62.4162	848.750	13.5982	848.750	13.5982	848.75	13.5982
4°	39°.2	62.4250	848.143	13.5866	848.139	13.5865	"	13.5936
5°	41°	62.4229	847.991	13.5846	847.986	13.5845	"	13.5968
10°	50°	62.4074	847.231	13.5758	847.222	13.5757	"	13.6002
15°	59°	62.3745	846.472	13.5708	846.458	13.5706	"	13.6073
20°	68°	62.3288	845.714	13.5697	845.695	13.5694	"	13.6184
25°	77°	62.2653	844.955	13.5724	844 981	13.5720	"	13.6334
30°	86°	62.1688	844.198	13.5791	844.167	13.5786	"	13.6523
35°	95°	62.0668	843.440	13.5892	843.408	13.5886	"	13.6748

Furthermore, to obtain the weight of 1 cu. ft. of mercury at other temperatures, as given in column 4, use was made of Regnault's formula for the cubical expansion of mercury,

$$v = v_o (1 + 0.000179007\, t + 0.0000000252\, t^2),$$

where v_o and v are the volumes at 0° and t° Cent., respectively; and to compare the results thus obtained with those derived from the simple approximate formula used by the authors, viz.: $v = \frac{v_o}{1 - 0.00018t}$, columns 6 and 7 have been computed. Column 9 gives the specific gravity of mercury at 0° Cent., relatively to water at the various other temperatures.

Mr. Kuichling. An examination of the last column shows that when the temperature of the water in the pipe is between 0° and 15° Cent., no appreciable error will be made by using the constant factor 13.60 (or the relative specific gravity of mercury at 0° Cent.) in order to convert a maximum height of only a few inches of mercury at 0° Cent. into the equivalent height of a column of water of somewhat higher temperature, as was done by the authors. If, however, the manometers were used to determine the difference in level between the two stations with the water at 20° Cent., for example, the use of the factor 13.60 instead of 13.6184 as per Table No. 10, would cause, with a mercury column 22.707 ft. high, an error of 0.418 ft. in the equivalent height of the water column, which would be intolerable in so short a length of pipe as 4 367 ft. It therefore follows that for exact leveling operations with open mercury manometers, either the correct value of the relative specific gravity of the mercury must be used, or the instruments must be carefully calibrated under conditions of temperature similar to those which prevailed during the observations. In general, for exact work, such a calibration is preferable to the use of the aforesaid theoretical factors or specific gravities, since account is thereby taken directly of any impurities in both the water and the mercury.

An instance of the use of an open mercury manometer to ascertain the difference in level between two points nearly 10.3 miles apart on a water conduit may perhaps be of interest in this connection. The experiment was carried out under the writer's direction in October, 1897, for the purpose of securing a check upon three sets of levels, which had been run at different times over the route of the conduit as correctly as possible with good wye-levels, and which did not agree with each other as closely as was deemed desirable for hydraulic observations. The difference in elevation of these two points was about 57 ft., and the elevations of the lower point given by the levels were +248.370, +249.230 and +248.550, the average of these three elevations being +248.717, and the difference between the first and second being 0.86 ft. The manometer was accordingly set up at the lower station several hours before readings were commenced, so that the mercury had ample time to acquire the same temperature as the air, and it was also placed so as to remain shaded; but as the day was cloudy, this precaution was not necessary. A good aneroid barometer was likewise provided at each station, in order to note whether any appreciable differences in the atmospheric pressure occurred. No such difference, however, was found. Observations were begun about half an hour after the outlet valve of the conduit was closed, and were continued at frequent intervals during a period of 46 minutes, in which the mean temperature of the air (and presumably also the mercury) was 7° Cent., while that of the water was 15° Cent., and the height of the mercury column was 4.170 ft. At these temperatures

the relative specific gravity of mercury is 13.5903, so that the corresponding height of the water column was 56.6716 ft.; and by using this value along with the other data pertaining to the case, the true elevation of the lower point becomes +248.788. On the other hand, if the simpler approximate method of reduction adopted by the authors had been followed in this case, the computed height of the water column would have been 56.6406 ft., thus making the required elevation 0.031 ft. less. Mr. Kuichling.

It may also be remarked that the foregoing manometric leveling operation was subsequently checked by a direct calibration of the gauge, with the mercury, water and air all at a practically uniform temperature of 7° Cent., the instrument with its attached thermometer being packed in cotton waste which was kept saturated with a small stream of water from a faucet. The relative specific gravity of the mercury thus found agreed, up to the fifth decimal place, with that which was computed in the manner set forth in Table No. 10, and hence the terminal elevation mentioned has been accepted as correct, the inference being that both the water and the mercury were practically pure and followed exactly the indicated laws of variation in weight per unit of volume with changes in temperature. It should also be added that the manometric work above described was supplemented, on the remaining 9.4 miles of the conduit, by the direct observation and measurement of the static head of water as its surface stood exposed in a piezometer vessel located on the hillside at the lower end of the section; but as the further details of this case are not pertinent to the subject under discussion, they are accordingly omitted.

In their summary of available data relating to riveted pipes, the authors reach the conclusion that "for conduits of 36 ins. diameter or over, it is unsafe to count on a value of c greater than 110 for new pipes"; and, "if there are frequent or sharp bends the coefficient will doubtless be still less." Without desiring to criticise this general statement, the speaker ventures to express the belief that by taking proper account of the various resistances to the flow of the water which are peculiar to riveted pipes, the values of the coefficient c will become much less discordant than they now appear. No good reason can be given why there should be more friction on a smooth asphaltic coating of a steel plate than on the similar pitch coating of a cast-iron pipe, or on the smooth and bare surface of a well-made wooden pipe; but when it is considered that the riveted pipe presents on its interior a vast number of projecting rivet heads and plate edges, all of which represent variations in the diameter of the conduit and consequent small losses of head; also that changes in direction must necessarily be made by a series of abrupt deflections instead of by continuous curves of large radius, it is not surprising to find a marked difference

Mr. Kuichling. in the value of c in favor of cast-iron or wooden pipes. If, however, these numerous small resistances or losses of head in a riveted pipe are properly estimated and deducted from the observed total head or fall, so as to place such a conduit on a comparable basis with a similar line of cast-iron or wooden pipe, it will doubtless be found that the differences in the resulting values of c will be quite small.

The truth of this statement is demonstrated by the following investigation of some of the new data submitted by the authors, and of certain other data relating to the new 38-in. riveted steel conduit of the Rochester water-works. The Ogden steel pipe is of practically uniform diameter throughout; its length of 4 367 ft. between the two manometer stations of the authors being presumably made up of about 469 straight sections of 110 ins. each, and 27 short beveled or angle sections of about 35 ins. each, as deduced from the description of the work by Henry Goldmark,* M. Am. Soc. C. E. Furthermore, these 496 sections have a mean internal diameter of 72.22 ins., and are connected by means of butted joints with broad external round straps 10.5 ins. wide, each having four rows of rivets, so that in said length there are at least 1 984 circular rows of rivets with heads projecting inside. It also appears that there are on the average 62 rivets in each row, the mean diameter of the rivets being 1 in., and the sectional area of the conical head of such a rivet being at least 0.80 sq. in., thus presenting at each row an aggregate internal obstructing area of not less than 50 sq. ins., which is equivalent to a continuous ring or internal fillet 0.22 in. thick. The 27 angle sections, moreover, each cause an abrupt deflection of 4.5°, so that the total curvature in the line of the pipe amounts to 121.5 degrees. Account should properly also be taken of the appreciable obstruction of 11 sq. ins. of sectional area caused by the internal straps and projecting heads of the six rows of rivets of the butted longitudinal or straight seams, but it is here omitted for the sake of brevity.

Now, from the elementary principles of hydraulics, as set forth by standard authorities, the losses of head caused by interior constrictions or diaphragms in a pipe, and by abrupt angular deflections thereof, as well as by sudden enlargements of diameter, such as occur when the ends of the cylindrical sections overlap instead of being butted together, are all susceptible of computation, and it is usual to express these losses in terms of the height or head due to the mean velocity of discharge. Unfortunately the necessary empirical data available for such computations have all been derived from experiments with small pipes, but as no others are at hand, their accuracy may be tested by applying them to the cases of the Ogden and Rochester conduits; and if the results so obtained yield ultimate values for the coefficient c, in the Chezy formula $v = c\sqrt{r\,s}$, which are in close

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

accord with the values of c for smooth cast-iron or wooden pipes, it must follow that such available data are approximately correct, and also that by their use many existing uncertainties and incongruities in the discharge of riveted pipes may be eliminated. Mr. Kuichling.

With this end in view, use has been made of the data afforded by experiment No. 28 of the authors, in conjunction with the other details above excerpted and deduced from Mr. Goldmark's paper. The Ogden pipe should, in the writer's opinion, be considered as practically made with four low constrictions or internal fillets (representing the equivalent sectional area occupied by the rivet heads) at every round joint, each of which produces a certain loss of head; and with reference to the notation indicated in Fig. 18, we will have the following elements:

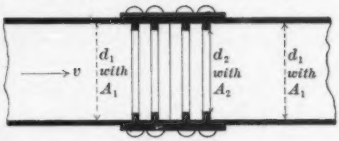

FIG. 18.

Mean internal diameter of pipe, $d_1 = 72.22$ ins.
Mean internal diameter of constriction or fillet, $d_2 = 71.78$ ins.
Ratio m of area A_2 to area A_1, $m = 0.98785$.
Number of fillets, $n_1 = 1984$.
Number of abrupt 4° 30′-deflections, $n_2 = 27$.
Length, $l = 4\,367$ ft.
Total observed loss of head, $h = 3.386$ ft.
Mean velocity of discharge, $v = 3.846$ ft.
Mean hydraulic radius, $r = 1.50458$ ft.
Observed hydraulic slope, $s = \frac{h}{l} = 0.00077536$.

Whence the deduced value of c, in $v = c\sqrt{r\,s}$, is $c = 112.60$. For the loss of head h_1, due to each constriction, all the standard textbooks on hydraulics give Weisbach's formula:

$$h_1 = \left(\frac{A_1}{\alpha\, A_2} - 1\right)^2 \frac{v^2}{2\,g} = \left(\frac{1}{\alpha\, m} - 1\right)^2 \frac{v^2}{2\,g},$$

in which α is an empirical coefficient varying with the ratio $m = \frac{A_2}{A_1}$ and capable of expression within the limits $m = 0.8$ and $m = 1.0$ by approximately: $\alpha = 1.225 + 1.45\,m^2 - 1.675\,m$, as deduced from Weisbach's figures by the writer. Substituting therein the above values of m, n and v, and taking $2\,g = 64.293$ sec.-ft., corresponding to the latitude of Ogden and its elevation of 4 307 ft. above sea level, there results $\alpha = 0.98533$, and the aggregate loss of head by 1 984 constrictions $n_1\,h_1 = 0.34196$ ft.

For the loss of head due to the 27 abrupt deflections of the angle sections, each of $\varphi = 4° 30'$, we must also have recourse to Weisbach's formula:

Mr. Kuichling.

$$h_2 = \left(0.9457 \sin^2 \frac{\varphi}{2} + 2.047 \sin^4 \frac{\varphi}{2}\right) \frac{v^2}{2g},$$

which gives $n_2 h_2 = 0.00909$ ft.

The total computed loss of head from these two sources of resistance to the flow of water through the pipe is therefore $n_1 h_1 + n_2 h_2 = 0.35105$ ft., or over 10 % of the total observed loss of head, so that the head left to overcome the friction on the asphalt coating of the pipe is $h^1 = h - 0.35105 = 3.03495$ ft.; and computing therewith the value of c, so as to bring about a fair comparison with the smooth wooden pipe or with a similar line of cast-iron pipe, we obtain $c^1 = 118.94$; whereas the omission of the above-named losses causes this coefficient to become $c = 112.60$. Now, from the authors' experiments Nos. 40, 44, 45 and 46 on the wooden pipe, in which the velocities were nearly as great as in experiment No. 28 on the riveted steel pipe, the average value of c is $c = 125.5$, and by rejecting the highest value of c in this series the average is $c = 123.67$; hence it follows that, by properly taking into account the aforesaid losses of head in the steel conduit, which are not present in the wooden conduit, the difference between the computed values of c is greatly reduced.

It may also be noted that this small difference can readily be attributed, both to the retarding influence of the corrugations or wrinkles in the asphalt coating of the steel pipe, and to the reduction of the sectional area of the pipe by the interior longitudinal strap and rivet heads already mentioned. Such corrugations appear to be unavoidable in the usual process of coating pipes by dipping them once or twice in a bath of melted asphalt, as will be seen from Figs. 1 and 2, Plate XL, which are from photographs of the interior of one of the 38-in. and 60-in. pipes of the new Rochester conduit; and if the relatively small constrictions made by the interior rivet heads of the round seams produce a loss of more than 10% of the total head, as above computed, it cannot be doubted that the effect of the vastly targer number of smaller constrictions represented by the corrugations of the coatng will become appreciable. It is therefore to be anticipated that the ultimate value of c in a steel pipe, coated in the usual manner by dipping in a bath of asphalt, and after eliminating all other special resistances as above indicated, will be somewhat less than in a smooth wooden pipe.

Proceeding next to the consideration of the first or upper section of the 38-in. Rochester conduit, the following elements are available from the construction data and the carefully made, first gauging of the discharge on October 4th, 1895, when the average age of the pipe was 1.79 years. In this case, however, all the round seams are lapped, and provided with only a single row of rivets, except for a length of about 200 ft., at every junction where the plate thickness is varied, the round seams then being made with two rows of rivets.

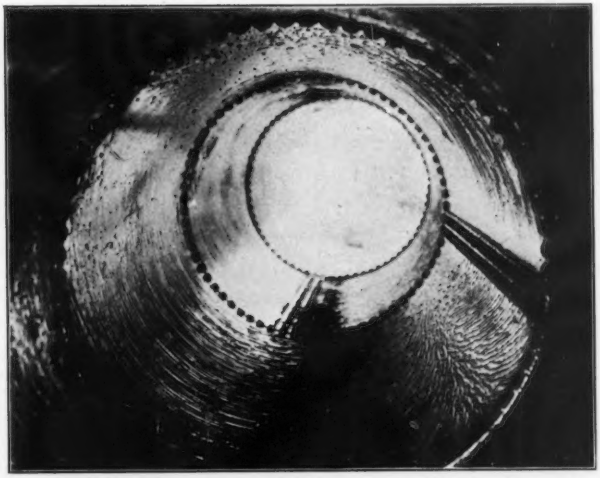

FIG. 1.

FIG. 2.

The general style of construction, as well as the notation, is indicated Mr. Kuichling in Fig. 19, and we have:

Internal diameter of inside courses or sections, $d_1 = 38$ ins.

Average plate thickness $= 0.27129$ in.

Average diameter of rivets $= 0.6676$ in.

Average sectional area of rivet head $= 0.35215$ sq. in.

Average number of heads in round seams $= 76.354$.

FIG. 19.

Average area occupied by interior rivet heads in each round seam $= 26.888$ sq. ins.

Equivalent thickness of constriction or fillet $= 0.2265$ in.

Resulting constricted diameter, $d_2 = 37.547$ in.

Total number of round rows of rivets or equivalent fillets, $n_1 = 14.143$.

Average internal diameter of outside courses or sections, $d_3 = 38.5426$ ins.

Total number n_3 of abrupt enlargements of diameter from d_1 to $d_3 = 6\,700$.

Ratio of area A_2 to area A_1, $m_1 = 0.97629$.

Ratio of area A_1 to area A_3, $m_2 = 0.97204$.

Total number of uniform deflections of $\varphi_1 = 2^\circ\ 43'\ 19''$ for horizontal and vertical curves is 914.

Same for $\phi_2 = 1^\circ\ 21'\ 51''$ is 96.

Same for average of $\phi_3 = 1^\circ\ 48'\ 48''$ for small irregular angles is 311.

Total length of pipe between points of observation, $l = 91\,640.83$ ft., of which 87.21 ft. consists of nominally 36-in. cast-iron pipe, special castings and stop-valves, whose actual least internal diameters range, however, from 36 to 38 ins., together with one 36-in. curve having a mean radius of 12 ft., and central angle $\phi_4 = 9^\circ$.

Total observed loss of head, including height due to velocity and entrance, $h = 91.053$ ft.

Mean velocity of discharge, $v_1 = 3.2733$ ft.

Mean hydraulic radius, referred to inside courses, $r_1 = 0.79167$ ft.

Observed hydraulic slope, $s = \dfrac{h}{l} = 0.00099359$.

Whence the deduced value of c in $v_1 = c\sqrt{r_1 s}$ is $c = 116.71$.

Furthermore, for the given latitude and elevation, $2g = 64.328$.

Computing as above the losses of head due to the constrictions and abrupt angular deflections, we find: $\alpha_1 = 0.97177$;

$$n_1 h_1 = n_1 \left(\frac{1}{\alpha_1 m_1} - 1\right)^2 \frac{v_1^2}{2g} = 6.87968 \text{ ft.};$$

and the aggregate loss by the aforesaid deflections, as well as the

Mr. Kuichling. cast-iron curve, $\Sigma h_2 = 0.09744$ ft. For the loss of head by the abrupt enlargements of diameter from d_1 to d_3, the usual formula is:

$$h_3 = \left(1 - \frac{A_1}{A_3}\right)^2 \frac{v_1^2}{2g} = (1 - m_2)^2 \frac{v_1^2}{2g},$$

from which there follows: $n_3\ h_3 = 0.87222$ ft.; and finally, for the velocity head and the loss of head caused by the passage through ten stop-valves and special castings, we have respectively, 0.16656 ft., and 0.13910 ft. The sum of all these losses of head is 8.155 ft., which being subtracted from the total observed head or fall, leaves for the available head to overcome the friction on the asphalt coating: $h^1 = h - 8.155 = 82.898$ ft. With this fall the corrected value of the coefficient c for the steel pipe, when divested of the several special resistances above enumerated, becomes $c^1 = 122.32$. Now, from two equally careful gaugings made on October 17th and November 7th, 1895, of the second section of the Rochester conduit, whose upper part consists of 924 ft. of 36-in. cast-iron pipe, the values of the coefficient c in the latter for velocities of 4.204 and 4.234 ft. per second were found to be, respectively, 129.45 and 125.25, the loss of head in a length of 889.57 ft. of said pipe being determined separately from open piezometers; hence in this case also, the difference between the values of c for cast-iron and riveted steel pipes becomes quite small when proper allowance is made for the losses of head peculiar to the latter kind of pipe.

It may also be mentioned that the above mode of computation was applied by the writer to a few other cases of new riveted pipe where accurate data have recently become available, with results similar to those above set forth; and he is therefore inclined to believe that the empirical coefficients deduced by Weisbach from experiments with constrictions in small pipes are approximately applicable to large pipes. At all events, the foregoing computations relating to the 6-ft. Ogden pipe and the 38-in. Rochester conduit, appear to confirm this view quite strongly. Experienced hydraulicians will, however, prefer to make use of coefficients derived from experiments on a large scale; and, as the subject is of great commercial importance, it is hoped that suitable investigations of the losses of head due to rivet heads, constrictions, deflections and abrupt enlargements, as indicated above, will soon be undertaken and published. Engineers will then be able to make much more intelligent analyses of complex hydraulic problems, and less room for error will occur in the choice of a proper "coefficient of roughness" for pipes. In fact, it is manifestly unfair to make use of such a coefficient except in cases where exactly similar conditions prevail, or where the influence of special disturbances has been eliminated, as was done in the preceding instances of new constructions. After incrustations of rust, sediment, organic growths, etc., have made their appearance on the interior of a pipe, other con

ditions arise which are probably altogether too complicated to admit of initial computation, and in such cases the results of practical experience alone must govern. Mr. Kuichling.

In illustration of these latter remarks, the writer cites the results of several subsequent gaugings of the 36-in. cast-iron pipe in the second section of the Rochester conduit, referred to above. As already stated, the values of the coefficient c found from the observations of October 17th and November 7th, 1895, when the pipe was still comparatively new, were, respectively, 129.45 and 125.25; while from the tests made on July 28th, November 11th and November 19th, 1897, and on June 4th, 1898, said coefficient was found to have fallen to 99.22, 66.84, 66.24 and 70.23, respectively; the cause of the great decrease probably being the profuse development of organic growths in the pipe at the intermediate storage reservoir, where both the pressure and the normal velocity of flow are quite small. In contrast with these values of c for the originally smooth and clean cast-iron pipe may be placed the corresponding values of c for the succeeding long line of 38-in. riveted steel pipe, which were, respectively, on the dates mentioned: 109.35, 109.07, 106.11, 107.11, 106.46, and 105.65. It should also be noted that these figures refer only to the second section of the conduit, in which the conditions differ somewhat from those in the first section previously considered.

In connection with the various resistances to motion in lap-jointed steel pipes, mention may likewise be made of the loss of head caused by the passage of the water from the "outside" to the "inside" courses, or from the section A_3 to A_1, Fig. 19. No computation of this loss was made by the writer, inasmuch as reliable empirical data therefor, with small differences of A_3 and A_1, are not yet available, also because such loss appears to be insignificant in comparison with the loss due to enlargement of section, or the passage from A_1 to A_3; and it is also probable that such loss is compensated by the slight advantage of flow gained in the larger sectional areas of the outside courses. Another important element is the deformation or flattening of a large riveted pipe made of thin plates, by the weight of the earth covering, in places where the internal water pressure is too small to insure a full cylindrical cross-section. While this point does not apply to the Ogden pipe, where the pressures are very great, it is nevertheless pertinent to several other riveted conduits cited by the authors, and is of appreciable influence in the resulting values of the coefficient c. Such a flattening produces a reduction of sectional area while the wetted perimeter remains constant, and hence causes a decrease in the value of the mean hydraulic radius r, which, if duly taken into account, increases the value of c, as shown by the following example:

The paper on "The Distortion of Riveted Pipe by Back-Filling," by D. D. Clarke, M. Am. Soc. C. E.,* cites the case of a new 42-in. steel

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 93.

Mr. Kuichling. pipe, whose original vertical diameter was reduced at some points about 4 ins. by the weight of the superincumbent earth. This reduction is nearly 10% of the initial diameter, and produces an appreciably smaller sectional area. To investigate the case generally, let it be assumed that the flattened circle becomes an ellipse with an area A_1, a constant circumference $\pi\, d_0$ and a vertical minor axis d_1, the diameter of the original circle being d_0 and its area A_0; also let $n = \frac{d_1}{d_0}$ and $m = \frac{A_1}{A_0}$.

Then

$$m = \frac{n}{5}(4 - 3\,n)\left\{1 + \sqrt{1 + \frac{n\,(40 - 25\,n)}{(4 - 3\,n)^2}}\right\}.$$

Furthermore, for any given discharge Q through the pipe,

$$Q = A_0\, v_0 = A_0\, c_0 \sqrt{r_0\, s}, \text{ or } Q = A_1\, v_1 = A_1\, c_1 \sqrt{r_1\, s},$$

according as the section is a circle or an ellipse; and since

$$r_0 = \frac{A_0}{\pi\, d_0} \text{ and } r_1 = \frac{A_1}{\pi\, d_0}, \text{ or } \frac{r_1}{r_0} = \frac{A_1}{A_0} = m,$$

there follows:

$$\frac{c_1}{c_0} = \frac{A_0 \sqrt{r_0}}{A_1 \sqrt{r_1}} = \frac{1}{m \sqrt{m}}, \text{ or } c_1 = \frac{1}{m^{\frac{3}{2}}} c_0.$$

Now, for a reduction of 10% in vertical diameter, or for $d_1 = 0.9\, d_0$, $n = 0.9$ and $m = 0.9857$, whence $\frac{1}{m^{\frac{3}{2}}} = 1.022$ and $c_1 = 1.022\, c_0$; thus showing that for such a deformation or flattening of the pipe the coefficient c_1 becomes 2.2% greater than for the truly circular pipe.

Doubtless the foregoing refinements of computation will appear trivial to many engineers who are accustomed to dealing with ample margins of safety in the design of public works, but they become necessary when comparisons are made, either with the view of establishing improved data for future guidance, or to arrive at fundamental principles, such as the proper value of the coefficient c for different classes of surface.

In conclusion, the writer desires to avail himself of this opportunity to correct a numerical error which was inadvertently allowed to appear in one of his published reports,* and which was quoted by the authors in their paper at page 504. The experiments referred to were made in June and October, 1896, on the lower half of the first section of the new Rochester conduit, the length of 38-in. riveted steel pipe here involved being 46 338.76 ft., or 8.776 miles, instead of 34 468 ft., or 6.53 miles, while the total fall or loss of head, as computed from manometric observations at the lower end, ranged from 0.922 to 8.391 ft., instead of 1.08 to 6.95 ft. For the whole series of experiments,

* Report of Executive Board of Rochester, N. Y., for 1896.

the value of c ranged from 69.496 to 112.154, with an average of 101.78 instead of 99.04; but by excluding the three lowest values of c, which relate to the three smallest velocities, namely, $v = 0.276$, 0.489 and 0.612 ft. per second, the average value becomes $c = 107.35$; furthermore, by taking into account only the five experiments which were of longest duration and wherein the velocities ranged from 0.758 to 1.223 ft. per second, the average value of c becomes $c = 105.12$. Mr. Kuichling.

It should be noted that this series of gaugings was made to find the discharge through the said portion of the conduit for different degrees of opening of the 8-in. by-pass valve on a 36-in. stop-valve located at the upper end of said portion, the large valve remaining closed. The various discharges and velocities in the conduit were computed from the observed rise of the water in the large storage reservoir at the lower end of said portion during periods of from 4.2 to 13.6 hours, with the addition thereto of both the observed evaporation from the water surface during such periods and the previously measured rate of percolation or leakage of the reservoir when all the inlet and outlet gates thereof were tightly closed, so as to obtain the gross delivery of the pipe. The greatest computed velocity, 1.223 ft. per second, occurred when the said 8-in. by-pass valve was wide open; and the resulting average values of c above mentioned may be compared with the following values of c for the entire length of 91 640.83 ft. of the first section of the conduit, as found at various times for full flow under existing conditions of restricted discharge at the reservoir: October 4th, 1895, $v = 3.2733$ ft. per second, and $c = 116.71$; December 23d, 1895, $v = 3.2308$ and $c = 114.05$; July 23d, 1897, $v = 3.2482$ and $c = 115.50$; November 18th, 1897, $v = 3.1481$ and $c = 112.04$.

To complete this correction of error in the report mentioned, it should also be added that the mean value of the coefficient c, there stated to have been found in 1895 for the entire first section of the conduit, should be $c = 115.38$ instead of 109.25, which latter refers to the second section. The details of these experiments were not published, owing to the discordances in the resulting values of c, which were apparently due to slight errors of observation with the mercury manometer, as in the case of the Ogden experiments; also, because it was intended to repeat the observations with the utmost care, and extend them over longer periods of time.

CORRESPONDENCE.

Mr. Adams. ARTHUR L. ADAMS, M. Am. Soc. C. E.—The great disparity in the results obtained from the experiments on the steel pipe, especially for velocities of less than 2 ft. per second, are discouraging to any attempt at analytical study. More particularly is this true because of the unlooked-for difficulty with the mercury gauge having cast some doubt upon the reliability of the results.

The figures obtained from the stave pipe experiments are much more harmonious for velocities exceeding $1\frac{1}{4}$ ft. per second. It would seem that a much greater length of pipe than 2 700 ft. might have been used to good advantage, especially since a direct measurement of heads could not be made by water piezometers, there being no better way of eliminating the effect of the usual uncertainties arising in the reading of the gauges and from other causes. The writer always feels disposed to give weight to such experiments, when conducted with reasonable care, nearly in proportion to the length of the line experimented on.

The results, as a whole, are, in a measure, surprising. Most recent experiments have tended to cause the lowering of previously conceived ideas of the probable values for c in the formula $v = c\sqrt{r\,s}$ when applied to single-lap riveted pipes of large diameter. Indeed engineers are probably quite prepared for the receipt of further evidence that the value of c in pipes of this type is nearly independent of the diameter and also of the velocity when the latter exceeds about $2\frac{1}{2}$ ft. per second; but, that a smooth cylindrical pipe without any abrupt deflections or internal projections, such as the stave pipe experimented upon, should give no greater values for c than those presented (values, by the way, which are considerably less than have frequently been obtained with clean cast-iron pipes of large diameter and from stave pipes of much smaller diameter), it is believed will scarcely be in accord with the previously conceived opinions of many engineers.

While the number of actual experiments heretofore made to determine values of c suitable for stave pipes are exceedingly limited, as has been pointed out by the author, yet the numerous experiments on open and closed channels having uniformity in section and in condition of inner surface as to roughness, though, perhaps, to a degree less perfect than usually exists in a stave pipe, have, in almost all cases, within the limits of customary use, shown an increase in the value of c with each corresponding increase in the value of r. The experiments described by the authors do not indicate that such a law of increase applies to pipe as large as 6 ft. in diameter.

The results will, doubtless, be of value in finally determining a formula which shall be better adapted to pipes of this class than any at present used, but the writer feels that final judgment in this matter

Mr. Adams.

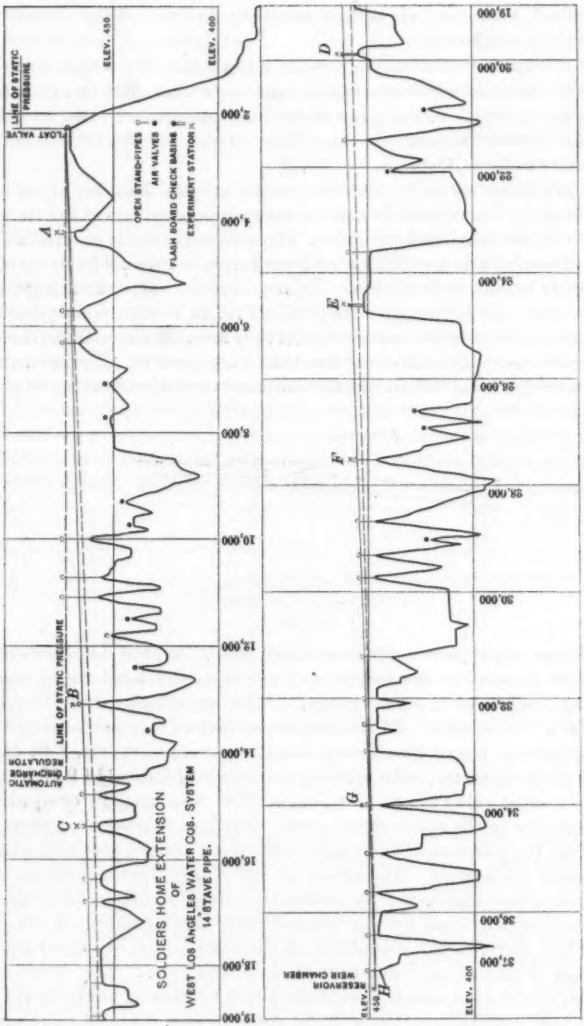

FIG. 20.

Mr. Adams. must be suspended until many additional experimental data have been accumulated. The full value of any experiments of this class yet conducted, since they are so comparatively few in number, cannot be ascertained until then.

As a slight contribution to such knowledge, the writer presents herewith the results of some experiments on a line of 14-in., redwood, stave pipe recently constructed under his supervision by the West Los Angeles Water Company for the supply of the Pacific Branch of the National Soldiers' Home.

In preparing for these observations the average diameter of the pipe was carefully determined by making many measurements of the circumference of the completed structure. These measurements covered all degrees of band spacing, which varied from 4½ ins. to 10 ins. The staves were uniformly $1\frac{5}{64}$ ins. in thickness. The net diameter varied from 13.99 ins. to 14.11 ins. By reference to the profile, Fig. 20, it will be seen that the vertical curves are quite numerous, but they are all made without the use of specials and with radii of not less than perhaps 40 ft. The horizontal curves are few, and 286 ft. was the minimum length of radius used.

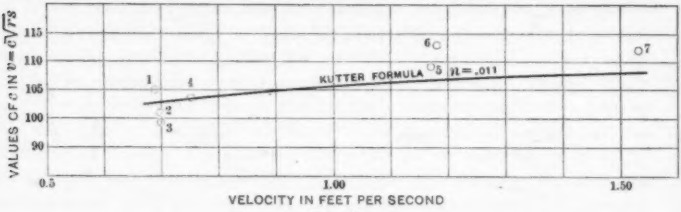

Fig. 21.

In these experiments the discharge was measured by means of a 4-ft. weir located at the terminus of the line; the head being measured by means of a hook gauge. The customary weir formula $Q = C L \sqrt{h^3}$ was used. In the fifth experiment a volumetric measurement of several hours' flow into a small concrete reservoir, 8 ft. deep and 50 ft. in diameter, with vertical inside walls, indicated 3.33 as being the correct value for C. The same value was used in computing the discharge in the other experiments. The loss of head was observed by noting the position of the water in the open stand-pipes and other designated structures. The effect of the slight fluctuations in the stand-pipes was eliminated by taking the mean of many readings, a suitable length of time having elapsed after the beginning of the experiment. The relative elevations of different points of observation, excepting A and B, were determined by a wye level. The points of uniform height at A and B were determined by closing a stop-gate at station 145, and allowing the water to come to rest. The outgoing pipe from the structure at D is enlarged to 16 ins. at the outlet. An allow-

ance for the velocity head only was here made. With the low velocities used, no sensible error could have thus resulted. Mr. Adams.

These experiments, classified as *A*, *B* and *C* in the order of their probable merit, are as follows:

TABLE No. 11.

No.	Section Experimented on.	Length of section-feet.	Velocity in feet per second.	Average pipe. Diameter. Inches.	Loss of head in 1 000 ft.	Resulting values of c in $v = c\sqrt{rs}$	Relative value of experiments.
1	2	3	4	5	6	7	8
1...	*C* — *D*	4 403	0.691	14.11	.145	105	*A*
2...	*G* — *H*	3 436	0.691	14.08	.161	101	*B*
3...	*D* — *E*	4 825	0.698	14.05	.170	99	*C*
4...	*A* — *B*	8 931	0.751	14.05	.178	104	*B*
5...	*F* — *H*	9 912	1.167	14.05	.391	109	*B*
6...	*A* — *B*	8 931	1.181	14.05	.375	113	*A*
7...	*F* — *H*	9 912	1.531	14.05	.638	112	*A*

It will be noticed that these experiments have been made on five different stretches of pipe, ranging in length from 3 436 ft. to 9 912 ft.; and that only low velocities could be secured, such as usually produce the most discordant experimental results. Considering these conditions it would appear that the resulting values of c are quite concordant.

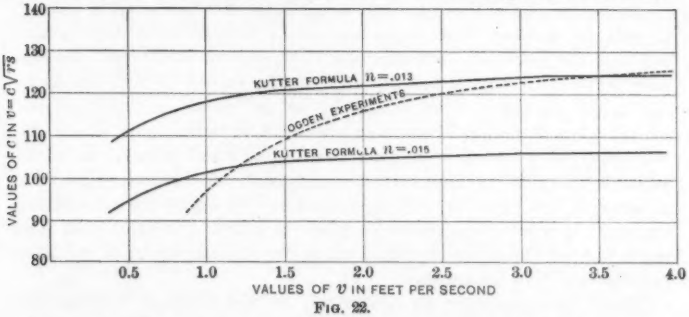

FIG. 22.

These values, while conforming quite closely to results obtained by the use of the Kutter formula, if n be taken at from .0108 to .011, for these velocities do not indicate .01 as being a safe assumed value for n, as have all previous experiments mentioned by the authors. Could higher velocities have been obtained, the results might then have shown that a lower value for n would have best fitted the results. This is indicated by the position of the observed values of c in relation to a curve given by the Kutter formula (Fig. 21). The same tendency of the experimental curve to depart from the Kutter curve in the low velocities is shown to exist in the Ogden experiments in Fig. 22.

Mr. Noble. T. A. NOBLE, M. Am. Soc. C. E.—The general conclusions arrived at by the authors, that the value of the coefficient c in the formula $v = c\sqrt{rs}$ is much less for wood pipes of the size experimented upon than had heretofore been supposed, and that the average value of n in Kutter's general formula for the flow of water in channels is about .013, will come as a surprise to the engineering public. The difference in the carrying capacity of wooden pipe, figured from this coefficient, would be great enough to seriously affect the capacity of a number of large conduits already constructed. It is, therefore, a matter of considerable importance that this question should be settled by a more extended series of experiments under, perhaps, more favorable conditions.

Several points which might have affected the results have occurred to the writer, and these are given with the object of getting the benefit of the authors' comments thereon.

It would seem that the distance between the points of observation on the wooden pipe was too short to obtain accurate results for the low friction heads and rates of discharge within the range possible in these tests. The greatest difference in head, in feet of mercury, is only 0.115. Any error due to reading the manometers or from any other cause would have a very much greater effect where the difference, as in this case, is small.

In the profile of the pipe line, as given in Mr. Goldmark's paper,* it will be noticed that there are about twenty-eight high points between the point of entrance and the point of discharge, whereas only sixteen air valves are shown. If the valves do not discharge air under pressure, they would be of no value for this purpose except when attended by some one detailed to open them frequently enough to keep the air from accumulating at these points. Is it not possible that during these tests a considerable accumulation of air was present in the pipe, in sufficient quantity to affect the rate of discharge? It would be germane to know if ample provision had been made to keep the pipes free from air at every point where it would be possible for it to accumulate.

There is apparently no uniformity in the relation between the average velocity in the pipe and the friction head, as determined by these experiments, except that, in general, the head increases with the increase in velocity. It is, therefore, doubtful if any conclusion as to the value of c and n in Kutter's formula, in any larger rate of discharge would be reliable. Table No. 12 gives the values of n for the different values of v, s and c, as determined from these experiments.

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 255.

TABLE No. 12. Mr. Noble.

Number of test.	v.	s.	c.	n.
35	.543	.0000401	69	.0204
47	.544	.0000251	88	.0160
41	.666	.0000551	73	.0199
31	.816	.0000201	148	.0100
48	1.217	.0000953	101	.0151
36	1.228	.000103	100	.0153
37	1.242	.0000952	104	.0148
42	1.332	.000106	106	.0145
31	1.482	.000111	115	.0135
49	1.876	.000181	114	.0138
43	1.978	.000211	111	.0141
32	2.138	.000211	120	.0132
38	2.218	.000235	117	.0135
50	2.536	.000300	119	.0134
33	2.689	.000331	120	.0133
39	2.783	.000361	119	.0134
51	3.149	.000452	121	.0132
34	3.453	.000517	124	.0129
40	3.568	.000566	122	.0132
46	3.617	.000501	131	.0123
45	3.627	.000547	126	.0127
44	3.645	.000576	123	.0130

There seems to be a decrease in the value of n with the increase in velocity. If this be true, it is possible that, when the pipe is discharging, up to the capacity for which it is designed, the value of n will be much less than that stated.

The change in the rate of discharge, which is constantly going on in a plant where power is distributed to consumers and where the load is constantly changing, must have a serious influence upon the apparent friction head, between any two points on the pipe line. A decrease in the rate of discharge would increase the head at the point of discharge. This effect becomes less, the greater the distance from the point of discharge, until it is zero at the point of entrance. The effect of an increase in the rate of discharge would have a corresponding but opposite effect upon the apparent friction head. Where a sufficient number of observations are taken it would seem that the errors from this cause would be neutralized. This, however, is not known, and hence the nature and value of this effect upon the friction head during these tests is unknown. Is this not the probable cause of the fluctuation in the relative value of v and s from which c (and n in Kutter's formula) are determined? In any case, it seems evident that, to get reliable and consistent results, the rate of discharge during each observation must be constant. Any acceleration or retardation of the rate of discharge would have a serious effect upon the friction head. This would be particularly true if the effect of retardation is not equal to the effect of acceleration, and if the rate of discharge at the end of the experiment were not equal to the rate at the beginning.

Mr. Weston. EDMUND B. WESTON, M. Am. Soc. C. E.—In 1889 the writer presented to the Society a paper* in which he gave his reason for considering Darcy's formula for the flow of water in pipes a very valuable formula for calculating the loss of head in pipes having interior surfaces similar to new cast-iron pipes; and he was much pleased in reading this interesting and valuable paper to note the small difference between 0.0216, the average experimental value of f obtained by the authors, as given in the paper, and 0.0202, which is a corresponding value of f deduced from Darcy's formula (reduced to English measures by the writer), for a pipe 72 ins. in diameter. The difference between the two is about 7 per cent. The authors state on page 498, that it was thought that the method adopted for measuring the loss of head should give results reliable to within 0.1 ft. in absolute value. The average of the losses of head given in column 8 of Table No. 2 is 1.342 ft., and 0.1 is about 7.5% of 1.342; whereas, the difference between the f obtained by the authors and that computed from Darcy's formula is, as given above, about 7 per cent. The writer recognizes the fact that the interior surface of a new, riveted steel pipe is more or less different from that of a new cast-iron pipe, and should expect the values of f for the two kinds of pipe to vary slightly, and, when computing results for steel pipes, he has been in the habit of increasing the value of f, deduced from Darcy's formula, by a small percentage, dependent in amount upon the character of the steel pipe and its joints. These percentages the writer determined from the results of a number of experiments made with steel pipes by different authorities, and he will still continue to make use of them, although the results given in the paper, considering the possible error mentioned by the authors, tends to show that Darcy's formula could have been advantageously used for determining the size of a pipe having similar characteristics to the 72-in. steel pipe used by the authors in making their experiments. For instance, on pages 492 and 493, the highest velocity given is 3.846 (No. 28), and the corresponding experimental loss of head per 1 000 ft. is 0.776. The loss of head per 1 000 ft. computed by Darcy's formula (as deduced by the writer), using the same velocity, is 0.774 ft., which is 0.002 ft. less than the experimental result. Or, again, calculating the loss of head by using 0.0216, the average value of f obtained by the authors, the result is a loss of head of 0.828 ft. per 1 000 ft., which is 0.054 ft. or about 7% more than that obtained by Darcy's formula; but within the limit of the possible error 0.1 mentioned by the authors on page 498.

Mr. Le Conte. L. J. LE CONTE, M. Am. Soc. C. E.—This paper is a valuable and timely contribution. The velocity coefficients for the 72-in. riveted pipe are somewhat less than many engineers would be willing to assume; yet they are such as every conservative engineer would natu-

* *Transactions*, Am. Soc. C. E., Vol. xxii, Jan., 1890.

Mr. Le Conte.

rally anticipate, basing his judgment on reliable data obtained from careful experiments with smaller pipes.

Riveted pipes have been extensively used on the Pacific Coast since 1870—the sizes running all the way up to 44 ins. in diameter and the thickness of the plate-iron from No. 14 (B. W. G.) up to $\frac{3}{8}$ in., while the velocity of flow varied from 1 ft. up to 10 ft. per second.

The general results obtained by practical experience go to show the paramount importance of the influence of the thickness of the plate-iron used. This factor is so important that the coefficient determined by experience on one pipe line is of no value whatever as applied to another line having the same diameter and slope. Hence it follows that the proper steps to be taken in any case, in order to select the most rational value for c, involves much more laborious calculation than is the case with any other class of pipe. With a riveted pipe line the velocity coefficient c changes materially with every change in the thickness of the iron, and before a satisfactory solution of the problem can be obtained the number of linear feet of No. 14 pipe, No. 12 pipe, No. 10 pipe, No. 8 pipe, No. 6 pipe, No. 4 pipe, etc., must be determined. This can best be taken from the profiles of the pipe line. Then, in the same manner as in the summation of moments, a value for the grand mean thickness of all the plate-iron used throughout the entire pipe-line can be deduced. This being completed, reference should be had to the Table of Coefficients (Table No. 13) submitted by the writer, and under the computed mean thickness and opposite the proper diameter may be found the most rational or true mean coefficient c, peculiar to the particular pipe line being considered.

It follows from the above that it is quite rare to find any two riveted pipe lines of the same diameter and slope which actually develop the same value for the velocity coefficient, because each pipe-line has its own particular value for c. Table No. 13 has been prepared by the writer for the purpose of facilitating the selection of a rational velocity coefficient for riveted pipe lines, using the old standard formula:

$$v = c\sqrt{r\,s}.$$

TABLE No. 13.—Coefficients Showing the Changes due to Different Diameters and Thicknesses of Iron in Riveted Pipe Lines.

Thickness—B. W. G.

Diameter in inches.	No. 16.	No. 14.	No. 12.	No. 10.	No. 9.	No. 8.	No. 7.	No. 6.	No. 4.	$\frac{1}{4}$ in.	No. 3.	$\frac{3}{8}$ in.
44.....	.....	.....	.....	120	119	117	116	114	112	111	110	100
42.....	.....	.....	.....	116	115	114	113	111	109	109	108	100
37½.....	.....	116	114	112	111	110	109	108	105	104	104	95
30.....	.....	114	112	110	109	108	106	105	102	101	100	91
24.....	.....	111	109	107	106	104	103	101	98	97	97	87
12.....	100	99	97	95	94	93	91	90	88	87	86	78

Mr. Le Conte. The authors refer to Table No. 4, and call attention to Kutter's sub-coefficient (n) increasing with the diameter. The writer would simply invite the authors' attention to the table of coefficients, herewith submitted, as redundant evidence of the fact that the coefficient (n) is a particularly active function in the case of riveted pipe lines, and that, unless full and complete information is furnished showing the extent and thickness of each and all the plate-iron used in manufacture, it is simply impossible to draw comparisons which have any value whatever.

The portion of this paper which is particularly interesting to the writer is that describing the experiments on the 72-in. wooden pipe, wherein the coefficient c varies from 100 to 126. The facts presented constitute a valuable addition to the knowledge on wooden pipe capacity and will prove to be a shock to many active and prurient advocates who have been claiming a safe and reliable value for $c = 166$, to be followed by an annual increase in value due to the "sliming up" of the interior surface of the pipe.

As to the changes in the value of the coefficient c with changes in the velocity of flow, experience on the Pacific coast seems to show that, for the larger sized pipes, the changes, if any, are very slight, but in the smaller sizes, especially when made of heavy plate-iron, the velocity coefficient c diminishes very materially with each increment of velocity of flow up to 10 ft. per second.

Mr. Henny. D. C. HENNY, M. Am. Soc. C. E.—The authors' experiments on the carrying capacity of these large wooden and steel pipes are of the highest interest. If they may be accepted as fairly correct it seems that the ultimate power-producing and earning capacity of the plant as a whole has been seriously overestimated. This is shown by a comparison between the values of c, as assumed by the designers, and those ascertained by the experiments of the authors. While the highest velocity experimented with was less than half of that on which the estimate of the ultimate capacity was based (8.75 ft. per second), the curves shown on Figs. 7 and 11 permit a rough approximate estimate of the values of c for said maximum velocity, which for steel pipe would be about 111 and for wooden pipe 130. As the respective values assumed by the designers were 120 and 160, it is evident that a deficiency of about 8% for steel pipe and of about 19% for wooden pipe may be found.

As far as wooden stave pipe is concerned, the adoption of the value $c = 160$ appears to have been justifiable. It is true that, owing to the comparatively recent introduction of this class of pipe, data which were directly available were almost entirely lacking, yet all the knowledge which existed pointed toward a value of c as high, or even higher, than that assumed.

In addition to the recent experiments quoted by the authors, there are available the results of three experiments of the Darcy & Bazin

series, Nos. 24, 25 and 26, which are fairly applicable, although they were made with open semi-circular channels and not with closed pipe. Where the depth in the former equals the radius there occur to the writer only a few points of difference between them and the closed pipes. Mr. Henny.

First.—The resistance of the air in contact with the water surface in semi-circular channels.

Second.—The possible occurrence of vertical curvature in the pipe.

Third.—The greater uncertainty regarding the actual path of each water particle in the pipe, the possible deviation from the hypothetical path being twice as much in a vertical direction as in the flume.

The first point of difference is in favor of the pipe, the other points are in favor of the flume, and all, so far as known, are small in ordinary cases.

The following table is offered as facilitating comparison:

TABLE No. 14.

Authority.	Author's No. of series.	Character of surface.	Form of cross-section.	Diameter in inches.	Fall per 1 000.	Observed velocity. Feet per second.	Coefficient c, Chezy form.	Coefficient n, Kutter form.
Adams	....	Planed wood.	Circle.	18	1.9628	3.605	132.9	.0099
Schuyler	....	Planed wood.	Circle.	30	.....	.....	.....	.0096
Darcy and Bazin.	24	Pure cement.	Semi-circle open at top. Depth of water-radius.	49.20	1.50	6.06	154.2	.0102
	25	3/5 cement. 1/5 fine sand.		49.20	1.50	5.55	141.7	.0109
	26	Partly planed wood.		55.08	1.50	5.54	133.5	.0118
Stearns	....	Coated cast iron, new.	Circle.	48	0.318	2.616	146.7	.0105
	....		.....	.....	0.711	3.738	140.1	.0110
	....		.....	.....	1.221	4.965	142.1	.0109
	....		.....	.....	1.849	6.195	144.1	.0108
Fitz-Gerald.	34	Same as above, 16 years old, tubercles removed.	Circle.	48	0.0689	1.113	134.1	.0107
	32		.....	.....	0.3234	2.472	137.5	.0111
	46		.....	.....	0.7167	3.723	139.1	.0111
	48		.....	.....	1.2414	4.973	141.1	.0110
	44		.....	.....	1.8283	6.141	143.6	.0108
Marx, Wing and Hoskins.	37	Planed wood.	Circle.	72.50	0.0952	1.242	104	.0148
	49		.....	.....	0.181	1.876	114	.0139
	33		.....	.....	0.331	2.689	120	.0134
	34		.....	.....	0.517	3.453	124	.0131

NOTE.—The serial numbers selected from the FitzGerald and the Marx, Wing and Hoskins experiments are representative of their respective series, and were chosen for their close coincidence, when platted, with the average curves presented by the experimenters.

Considering that planed wood offers less resistance to flow than any of the other surfaces mentioned, a comparison on the basis of the Chezy formula shows that the values of c obtained for the Ogden wooden pipe are remarkably low. From the Darcy and Bazin, Stearns

Mr. Henny. and FitzGerald experiments it would be concluded that, for a velocity of 6 ft. per second in a 4-ft. wooden-stave pipe, a value of $c = 155$ might well be expected, and in view of the tendency of c to increase, both with the diameter and the velocity, the designers in assuming a value of $c = 160$ for a 6-ft. wooden pipe, and 8.75 ft. per second velocity, showed no particular lack of conservatism.

The previous experiments with 18-in. and 30-in. wooden-stave pipe afford a less direct means of comparison on the basis of the Chezy formula owing to the great difference in diameter. If comparison is made on the basis of the Kutter formula it is found that even the lowest value of n deduced from the experiments on wooden-stave pipe made by the authors is much higher than it was reasonable to expect. The authors have, however, called attention to the fact that their results, both with wooden and steel pipe, lead them to question the reliability of this formula for large pipe. While it must be admitted that the variation in the values of n for varying velocities is large, yet the unquestioned wide applicability of the Kutter formula to open waterways of all dimensions and the fairly satisfactory results obtained, in applying it to pipes with diameters up to 48 ins., appear to necessitate stronger evidence than has so far been adduced before discarding its use in connection with diameters above 48 ins., although results such as have been obtained with the Ogden pipe may compel the allowance of greater margins of safety.

That the results obtained were not in accord with previous experience must have become apparent to the authors at an early stage of their experiments, and must have had the effect of rendering them more than usually cautious in guarding against errors in every direction. If the writer, therefore, further on, touches upon some elementary factors entering into the problem, it is with the expectation that the authors are fully prepared to furnish such additional information and data as may be deemed necessary to remove all reasonable doubt as to the thorough reliability of the results.

The authors have set forth with admirable clearness certain elements affecting the accuracy of their work. As is shown by them, the values determined from low velocities, say below 1 ft. per second, are unreliable. Comparing the values found in the wooden pipe experiments, for velocities above 1 ft. per second, among themselves, it is found that the variations from the average amount to about 6% both ways. Variations similarly determined from the FitzGerald experiments on 48-in. clean cast-iron pipe* amount to 1.3%, the length of pipe experimented with, expressed in diameters, being about the same and the range of velocities being greater. The unfavorable condition under which the authors labored, in respect to the drop of the grade line at both the upper and lower station with

* *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 261.

each increase of velocity, probably contributed to this result. The effect of this would have been less serious had the upper station been nearer the intake and had the section itself been longer. This is so evident that it may be presumed that serious obstacles prevented the use of the long upper section of the pipe above the tunnel for experimental purposes. Aside from the errors resulting from inaccuracies in observation, it may be well to consider possible errors not mentioned by the authors and affecting all observations alike. Mr. Henny.

The measurement of the length of pipe between the upper and lower stations leaves probably nothing to be desired. The determination of the average diameter must, however, have been a much more difficult matter, as the pipe was buried for almost its entire length. It has not been stated how the value, $72\frac{1}{2}$ ins., has been ascertained. From the dimensions of the bands and staves, as given in Mr. Goldmark's paper,* the writer would have concluded, in the absence of actual measurements after completion, that the diameter would most likely lie between $72\frac{1}{4}$ and 72 ins. The designers themselves, in computing the capacity, assumed the diameter at 6.03 ft., or about $\frac{1}{8}$ in. less than the authors. The length of the bands and the available thread permit a reduction of diameter to $71\frac{3}{4}$ ins., in cinching, which would be a reduction in circumference of less than 0.9%, starting from $72\frac{3}{8}$ ins., a larger proportionate reduction having been frequently measured by the writer on other wooden-stave pipe, particularly where the spacing of the bands is as close as on the pipe under discussion. This computed minimum diameter for dry pipe may suffer an additional reduction from the swelling of the staves which meets with resistance in every direction except toward the center of the pipe. Unless the interior diameter has been derived from actual measurements the possible error from this source exceeds 2 per cent.

The presence of air at the summits may have a seriously disturbing effect on the grade line, and has probably been carefully guarded against. The relief shaft at tunnel No. 7, if connecting with the tunnel at its top, may be expected to free the water of nearly all air. Yet, wherever, a summit occurs there exists danger of air accumulation, and in view of the presence, between the upper and lower stations of three summits and one practical summit at station 250, some positive evidence of absence of air, at these points, is required to allay doubt.

The omission, from the profile given by Mr. Goldmark,† of air-valves at some of these points may have been accidental, except possibly at station 250, but it should further be noticed that such air-valves as are described are not automatic, and remain closed while under pressure. In the account of the steel pipe experiments mention was made of careful manipulation being needed to keep the long pipe

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 255.

Mr. Henny. connecting the steel main with the manometer at the lower station wholly free from air. Did this indicate the presence of air in the water?

The writer has been informed that the Ogden River, during certain seasons of the year, carries a considerable proportion of sand. This, taken in connection with the absence of a reservoir at the upper end, implies the danger of sand having been carried into the pipe, unless a thoroughly effective sand trap was provided. The maximum velocity in the pipe used in experimenting was capable of transporting sand for long distances, and all the circumstances, unless wrongly understood by the writer, point toward the possibility of the presence of sediment in portions of the pipe line. Nor could the 6-in. gates which occur at depressions be in any way depended upon to flush such sediment out except for a few feet either way from the points where they are attached. In first opening they would enable some idea to be formed regarding the presence or absence of sediment in the pipe, otherwise they can be useful only as drain gates.

It will be observed that the errors, due to the above-mentioned sources, would not have the effect of neutralizing each other, but that they would all tend toward reducing the velocity and the computed value of c, and that some of them, unless their absence has been fairly established, may have a very important, if not a vitiating effect upon the results.

The proper method of attachment of a piezometer has been fully discussed by the authors. One of the advantages of a multiplicity of connecting points with the main pipe is that the effect of any imperfection of workmanship, at any one point, is lessened. As it was deemed sufficient to have only a single point of attachment and as very small losses of head had to be measured, especial care became necessary to comply with the well-known requirements, that the hole be smooth and the channel leading away from it have a direction accurately perpendicular to that of the flow. It was stated that in connecting the manometer to the wooden pipe a hole was bored at right angles to the axis of the pipe, that a pipe was screwed into this hole, and that care was taken that there should be no projection beyond the inner surface. If, as may have been the case, this connection was made without first draining the pipe or relieving the pressure, it must have been a difficult matter to ascertain the true condition of the hole, and to insure the absence of ragged edges and projecting slivers; so that some further particulars on this point are desirable.

The plan of the pipe line under consideration (Fig. 9) shows a long curve at the upper end and a short curve further down, the aggregate length being about half that of the entire line. The profile, moreover, indicates a succession of vertical curves so that the aggregate length of the actual tangents, as compared to that of the curves and the greatest length of any one tangent is evidently quite small, especially if expressed

in diameters. In this respect conditions differ from those under which the earlier tabulated experiments were made. Whether the effect of such a succession of easy curves upon the grade line is at all appreciable must remain a matter of speculation until it shall have received some attention from experimenters. Practically nothing is known of the path described by individual particles of water, either in a straight or in a curved pipe. It is difficult to conceive that a motion parallel to the axis of the pipe, as assumed in theory, could be even approximately maintained in a pipe curving both horizontally and vertically and it is highly probable that the relative position of the particles in the successive cross-sections is constantly changing. Under favorable conditions of curvature and velocity, as yet entirely undetermined, this change may acquire a certain degree of regularity, causing the advancing body, as a whole, to acquire a rotating motion as has been observed in hydraulic mining, when the pipe leading to the nozzle is curved. It has been found that a rotating motion may at times become sufficiently pronounced to be troublesome, and cause the water to whirl as it comes out of the nozzle, thus interfering with the erosive effect of the stream. The difficulty is overcome by placing thin partition plates in the pipe a short distance back of the nozzle. This is mentioned as having possibly affected, to a slight extent, the frictional losses measured, and to show that this subject is worthy the attention of experimenters. It does, however, in no way account for the great difference between the anticipated values of c and those found by the authors. Mr. Henny.

CHARLES D. MARX, M. Am. Soc. C. E.; CHARLES B. WING, Assoc. M. Am. Soc. C. E., and LEANDER M. HOSKINS, Esq.—The foregoing discussion and correspondence deal partly with the question of the reliability of the experimental results given in the paper, and partly with other matters relating to the carrying capacity of large pipes. In the further discussion now offered, it seems desirable to keep such considerations as bear directly upon the question of reliability distinct from the other and more general questions that have been raised. Messrs. Marx, Wing and Hoskins.

RELIABILITY OF RESULTS.

In considering the degree of reliability of a series of experimental results such as that under discussion, three different kinds of evidence may be available. The first relates to the conditions under which the observations were made and the methods adopted, the second to the degree of accordance of the results among themselves, and the third to their agreement or disagreement with the results of other similar experiments. Of these classes of evidence the first two are always of fundamental importance, and especially is this true in a case like the present, in which the other data available for comparison are very meager. In preparing the paper, it was the aim of the

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authors to describe fully and clearly the conditions and methods; to present the results in a form showing clearly their degree of accordance; and to present such data of similar experiments as seemed to be of direct value for purposes of comparison. In view of the questions that have been raised, some further consideration of these three phases of the subject seems desirable.

(*1*) *Conditions and Methods.*—Some of the questions raised relate to matters which are fully treated in the paper. In addition to what is there given regarding the conditions under which the experiments were made, the apparatus used, the methods of observation, and the method of reducing the results, the following may be said by way of direct reply to the questions raised in the discussion and correspondence:

It has been suggested that certain constants used in the work were not determined with sufficient precision. In particular, the specific gravity of the mercury used, the calibration of the gauges, and the diameter of the pipe have been called in question.

It is obvious that whatever error there may have been in the value of the specific gravity of the mercury affected only the value of the loss of head between piezometer stations. The authors felt justified in assuming that the value used (13.6) could be depended upon as correct to a degree greater than the possible degree of precision attainable in the observations. Table No. 10, given by Mr. Kuichling, shows that, so far as the effect of variations of temperature is concerned, the assumption is justified. It may be pointed out that, if the true value of the specific gravity is between 13.54 and 13.66, the error involved in the value 13.6 is less than $\frac{1}{2}$ of 1 per cent. An error of $\frac{1}{2}$ of 1% in the value of the lost head would affect f by the same relative amount, and c by half that relative amount, or $\frac{1}{4}$ of 1 per cent. Such an error is far within the possible limit of accuracy of the observations.

With regard to the calibration of the piezometers, it would appear that, since the position of the top of the mercury column was read directly by means of a fixed scale, the only calibration needed was the testing of the graduations of the scale and the determination of the height of the zero of the scale above the surface of the mercury in the reservoir. The degree of precision needed in the latter determination was not great, for reasons stated in the paper. The scales used were accurately graduated by a reliable maker, and were undoubtedly correct to an even greater degree of precision than the nature of the experiments necessitated.

The diameters of the steel and wooden pipes were not determined by direct measurement. The conditions made this impossible, even had it been desirable. The diameters were computed from the dimensions of the plates used (in case of the steel pipe) and of the staves

(in case of the wooden pipe). As the diameter of the wooden pipe especially has been questioned, it may be said that the value 72.5 ins. was taken from the data given in Mr. Goldmark's paper.* Messrs. Marx, Wing and Hoskins.

"The pipe was built of thirty-two staves, the finished staves being $7\frac{1}{2}$ ins. wide on the outside, $7\frac{1}{8}$ ins. wide on the inside and 2 ins. in thickness. * * * Both sides of the staves were dressed in a planing machine to conform to the outlines of a circle; the outside to a circle of $38\frac{1}{4}$ ins. radius, and the inside to a circle of $36\frac{1}{4}$ ins. radius."

If, instead of taking the value here given for the inner radius, $36\frac{1}{4}$ ins., the radius is computed from the width of the staves, a slightly greater value results. The value of the diameter thus computed is 72.57 ins. or 72.69 ins., according to whether the width $7\frac{1}{8}$ ins. was measured along the circular arc or along the chord. It thus appears that the dimensions of the staves give a value of the diameter a little greater than the value used by the authors, rather than a little less, as stated by Mr. Henny. As regards the effect of the swelling of the wood upon the diameter, the authors can express no opinion, having no experimental knowledge. Neither can definite information be given as to changes in the shape and size of the cross-section due to earth pressure.

Two remarks seem pertinent in this connection: *First*, whatever uncertainty exists as to the value of the diameter of the stave pipe when in use, similar uncertainty will exist in case of any pipe line to be designed in the future. *Second*, the greatest possible error that can be attributed to this uncertainty is insignificant in comparison with the difference between the results shown by the Ogden experiments and those hitherto accepted by some engineers.

The question has been raised as to whether the observations may not have been vitiated by the presence of air in the water. There are two ways in which, conceivably, air may influence measurements of the kind under discussion—by accumulating at the summits of the pipe line, thus reducing the cross-section at these points, and by accumulating in the piezometer pipes. As to the former, it may be said that the air-valves along the entire pipe line were opened every morning for the escape of whatever air might have accumulated. Although it is true that air-valves are not present at all the summits of the wooden pipe, the slope of this pipe is everywhere so small that the accumulation of air seems improbable. Moreover, as remarked by Mr. Henny, the relief shaft at the tunnel would have the effect of freeing the water from air just before it entered the section of wooden pipe experimented on. It may be said further that if air in this pipe had been a serious disturbing factor, the results would have shown much greater irregularity than is indicated in Fig. 10. As to the possible effect of air in the piezometer connections, the authors fully recognized the necessity of keeping these connections free from air. As stated in the

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 268.

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paper, an air-cock was in each case placed at the highest point of the pipe leading from the pressure-section to the piezometer, and this was frequently opened for the escape of air.

It may be said that, in the manipulation of the air-cocks, no noticeable amount of air was in any case found to have accumulated. That a small quantity did collect when the gauges were out of use for several hours was shown by the fact that slightly different gauge-readings were sometimes obtained before and after the cock was opened at the beginning of a series of observations.

The question as to the probability of sediment having been present in the pipe in quantities sufficient to affect appreciably its carrying capacity has been answered by Mr. Goldmark. It may be said further, that twice during the period covered by the observations the interior of the steel pipe was examined throughout its entire length, and that no important amount of sediment was found.

In reply to Mr. Noble's question regarding variations in the rate of discharge during the periods of observation, reference may be made to what is said under the heading "Plotting of Reduced Readings," page 480, and to the sample curves in Figs. 4 and 5. The steadiness of the rate of discharge is shown by the curves giving difference-gauge readings. At the time when the experiments were made, the consumption of water in the supply of power to customers was not subject to frequent variation, and the total rate of discharge could be adjusted, within certain limits, to the needs of the experiments.

The reason a longer section of the wooden pipe was not used in the experiments was on account of the existence of the open stand-pipe at the tunnel, permitting an overflow of water until the velocity reached about 1.5 ft. per second. It was desired to extend the observations to as low velocities as practicable, especially as, under the conditions then existing, the range was necessarily limited to velocities below 4 ft. per second.

In reply to Mr. Henny's question as to the attachment of piezometers to the wooden pipe, it is to be said that it was necessary to tap the pipe when under pressure, and that it was not practicable to insure perfect smoothness of the hole and freedom from splinters. That no serious splintering occurred was indicated by the smoothness of the issuing jet. The authors see no reason to believe that the presence of splinters at the inner edge of the hole would have an appreciable effect upon the indications of the piezometer.

Some remarks may be made by way of general comment upon the conditions under which the experiments were made, and upon the methods adopted. If comparison be made with similar experiments made elsewhere, it will be seen that the Ogden experiments were made under conditions some of which were favorable and others somewhat unfavorable.

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The conditions for measuring the rate of discharge were exceptionally favorable, owing to the presence of the Venturi meters. The reliability of these measurements has not been questioned in the discussion. It is believed that, where Venturi meters are available, most engineers will prefer this method of measurement, both to the use of weirs and to direct measurement in a reservoir. While weir measurements of the most careful kind will doubtless yield results which many engineers would regard as fully equal in reliability to those obtained with Venturi meters, it is probable that this high standard has been reached in only a few of the weir measurements made in experiments on the carrying capacity of large pipes.

As regards the measurement of lost head, where the pressure conditions permit the use of water piezometers, these are to be preferred to mercury gauges, since their indications are more precise in the ratio of the density of mercury to that of water. The experience of the authors leads them to believe, however, that if the mercury gauge is read to the thousandth of a foot, the last figure will usually be correct to within one unit. An error of 0.002 ft. would be equivalent to less than 0.03 ft. of water.

It is true that a great length of pipe is to be preferred to a small length, other conditions being equally favorable, since a given error in the measured loss of head is important in inverse ratio to the length. It may be suggested, however, that the use of the long pipe may not be without its disadvantages, among which may be mentioned the increased difficulty of avoiding the accumulation of air at summits.

It should be observed that the sacrifice of precision involved in the use of mercury gauges instead of water columns, and in the use of a comparatively short length of pipe, may be compensated by multiplying the number of observations. The observational errors in the values of the lost head are true "accidental" errors, as likely to be plus as minus. The importance of these errors, as affecting the mean of the series of results, is indicated by the degree of accordance of the results among themselves. This question will be discussed below.

The most unfavorable feature of the experiments was the leakage in the mercury gauge at the lower end of the steel pipe. This has been fully discussed in the paper, and needs no further comment here. The authors would only remark that, although the steel pipe observations show, on this account, less accordance than was expected, the general results are believed to be fairly reliable. Their reliability, in fact, has been accepted practically by those who have discussed the paper.

(2) *Accordance of Results.*—Some light is thrown upon the degree of precision of the results by a careful examination of their accordance among themselves.

It has been pointed out that the values of c for low velocities show very considerable discrepancies. The values of c do not, however,

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give direct indication of the degree of precision of the measurements. This is to be ascertained by comparison of the values of the quantities directly measured rather than of quantities computed from them. The values of the lost head, being obtained by taking differences of observed quantities, may be treated as if obtained by direct observation, and their accordance gives a direct indication of the precision of the observations; while the values of c, which vary inversely with v^2, magnify the errors very greatly for small values of v.

Referring first to the steel pipe observations, the best indication of the degree of accordance is afforded by Fig. 6. It is seen that the observations at low velocities show about the same accordance as the rest of the series.

If the curve shown in Fig. 6 be taken as representing the mean values resulting from the series of observations, the probable error of an observation may be estimated by computing residuals from the mean values, and applying the usual formula. These residuals are found by subtracting from each "observed" value of H' the value given by the curve for the same velocity. Applying the least-squares formula, the probable error of a single observation is found to be 0.0142. This is the probable error of an observed value of the loss of head per thousand feet. Multiplying by the length of the pipe expressed in thousands of feet, we find 0.062 ft. as the probable error of an observed value of the total loss of head.

The error in c caused by a given error in H' may be found by differentiating the formula $v = c\sqrt{rs}$, which may be written

$$c = 25.8\,\frac{v}{\sqrt{H'}}.$$

If δc is the error in c caused by an error $\delta H'$ in H', we have

$$\delta c = -\frac{12.9\ v}{\sqrt{H'^3}}\,\delta H'.$$

Using this relation, and assuming 0.0142 as the probable error of H', the values of the probable error of c for different values of v are as follows:

v	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
P. E. of c	59	15	6.5	3.7	2.3	1.6	1.2	0.91

It should be noticed that these values denote the probable error of the value of c derived from a single observation, not that of the adopted mean values. Of the degree of precision of these mean values, no very definite estimate can be made, since the values of H' given by the curve in Fig. 6 for different values of v are not obtained by averaging several direct observations on a quantity whose true value is constant, and no

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accurate estimate can be made of the number of observations contributing to the average result adopted for a given value of v. The mean values must, however, be regarded as having a decidedly higher degree of precision than a single observed value.

The sinuous character of the mean curve showing values of c (Fig. 7) has been referred to as casting suspicion on the results. As stated in the paper, the authors do not regard the experiments on the steel pipe as sufficiently precise to exclude the possibility that this sinuosity may be due to uncompensated errors. It may be remarked, however, that such a variation of c with the velocity cannot be regarded as extremely improbable *a priori*, especially if c is nearly constant, as seems to be the case for considerable ranges of velocity in large riveted pipes. A constant value of c would mean that the loss of head varies as the square of the velocity. This law would give, in Fig. 6, a parabola with vertex at the origin and axis vertical. It is entirely possible to assume a curve intersecting a given parabola in several points, without implying any irregularity in the law connecting H' and v. The curve shown in Fig. 6 implies a fairly regular law, notwithstanding that the corresponding curve for c is sinuous.

If the precision of the observed values of H' for the wooden pipe be estimated in the above manner, adopting the mean curve shown in Fig. 10, the results reached are the following:

Probable error of $H' = 0.0123$.

Probable error of an observed value of the total loss of head between pressure stations $= 0.0123 \times 2.71 = 0.033$.

If the very discrepant observation No. 46 be omitted, these values are reduced to 0.00802 and 0.022. Adopting the larger values, the probable error of the value of c obtained from a single observation, computed as in the case of the steel pipe observations, has the following values for different velocities:

v.....	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
P. E. of c.....	21	9.5	5.4	3.5	2.5	1.9	1.4	1.1

Referring to Mr. Noble's statement that the experiments appear to show no uniformity in the relation between mean velocity of flow and loss of head, it may be pointed out that, in the case of the experiments on the wooden pipe this relation appears to be well represented by the equation

$$H' = 0.0314\, v + 0.0343\, v^2.$$

The fact that this relation holds, within the range of the experiments, does not, of course, warrant its application to higher velocities.

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(3) Comparison with Other Experimental Data.—Many data relating to the carrying capacity of large riveted pipes are available. Although a considerable gap exists between the size of the Ogden pipe and the sizes of the largest pipes previously experimented on, the results appear to be accepted as in agreement with what previous knowledge would lead us to expect.

In the case of wooden pipe very little experimental knowledge is available for comparison. Reference is made in the paper to three experiments, and to these Mr. Adams has now added a fourth. The published accounts of the gaugings of the Denver and Butte City conduits are very meager, and without further information as to the methods used, the results do not appear to be entitled to great weight.* The diameters of these conduits are 30 ins. and 24 ins., respectively, while those of the pipes experimented upon by Mr. Adams at Astoria and at Los Angeles are 18 ins. and 14 ins., respectively. Experiments with pipes of these sizes, however reliable, do not afford a safe basis for estimating the capacity of a conduit 72 ins. in diameter.

It is true that in the absence of experimental knowledge obtained directly from wooden pipes, many engineers have felt justified in using the results of experiments made upon smooth cast-iron pipes, and upon open conduits with comparatively smooth surfaces. Even on this basis not many data are available for estimating the capacity of pipes as large as 6 ft. in diameter. So far as known to the authors, the experiments of Stearns and FitzGerald are the only careful gaugings made of smooth pipes of diameters as great as 4 ft., previous to those at Ogden.†

The curves given by Hamilton Smith, Jr., M. Am. Soc. C. E., showing the relation between c and v for different diameters up to 8 ft., appear to be, for the larger sizes, mainly speculative, obtained by assuming that the laws observed to hold for small pipes may be expected to hold for large sizes also. Such an extension of a purely empirical law far beyond the limits of the data upon which it is based can afford no reliable basis for estimating the credibility of a new set of experimental results.

To reason from cast-iron or cement-lined pipes to wooden pipes involves the assumption of equal "smoothness." To reason from

* Mr. D. C. Henny informs the authors that the Butte City gauging was made by him, and that the method used in measuring the velocity of flow was not such as to insure a high degree of accuracy. In regard to the Denver experiment, the statement of Mr. Schuyler, in the paper already cited, that the discharge measurements were checked by the use of a current meter, indicates that the standard of accuracy was not high.

† In the translation of the work of Ganguillet and Kutter, by Hering and Trautwine, the result of an experiment, by Jas. M. Gale, on a new cast-iron pipe 4 ft. in diameter, and 3¾ miles long, is given. The value of c is 112.4 for a velocity of 3.458 ft. per second, the corresponding value of n being 0.0168. It will be noticed that this value of c is very decidedly lower than those found by Stearns and FitzGerald. The method of measurement is not stated, and the degree of reliability of the results is unknown.

open conduits of various shapes to a closed circular pipe involves the assumption that the effect of the form of the cross-section upon the loss of head is wholly accounted for in the value of the hydraulic radius. In the opinion of the authors neither assumption can be justified theoretically, or (in the present state of knowledge) experimentally. The words "rough" and "smooth," when applied to the inner surface of a conduit, must be understood as referring to the total effect of the inner surface in retarding the flow. Whether the adhesion of water is greater or less for a surface of water-soaked wood than for an asphaltum-coated surface of cast iron can be known only by experiment. Messrs. Marx, Wing and Hoskins.

Causes of Loss of Head.

Among the questions considered in the correspondence is that of the causes of the losses of head observed in the steel and wooden pipes. No attempt was made by the authors to estimate, separately, the effects of the various conditions which obviously contributed to the total losses observed. The values found for H', c and f are to be regarded as resultant values, due to the combined effect of all resistances.

Mr. Kuichling's method of estimating the loss of head due to rivet heads or other protuberances is to be justified, if at all, by comparison with experimental results, not only in this case, but in many cases. The Weisbach formula, upon which the method is based, must be regarded as empirical, and this application appears to be a very radical extension of the limits within which the formula is to be used.

Mr. Le Conte suggests that the value of c for the steel pipe is a function of the thickness of the plates of which the pipe is made. In the case of lap-jointed pipes, whether "taper" or "cylinder," it is reasonable to suppose that the thickness of the plates is one of the factors governing the loss of head, but it is not evident that this is the case when butt joints are used, as in the Ogden pipe.

It may reasonably be assumed that a portion of the loss of head, in both the steel and the wood pipes, is due to bends. The amount of this effect is a matter of great interest and importance; but in the present state of experimental knowledge it seems useless to attempt a quantitative estimate. The authors do not regard it as improbable that a straight pipe, otherwise similar to the wooden pipe experimented upon, would give values of c materially higher than those found. As a guide to future design, however, the results do not lose value by reason of the bends, since in practice the existence of bends will be the rule rather than the exception.

Messrs. Marx, Wing and Hoskins.

Applicability of Kutter's Formula.

In the opinion of the authors, no rational formula for the discharge of pipes is in existence, and the complexity of the conditions affecting loss of head is such as to offer little ground for the belief that such a formula will be discovered in the future. So far as theoretical basis is concerned, the Chezy formula is as satisfactory as any.

The Kutter formula is merely an attempt to express the value of c in the Chezy formula as a function of three quantities, "roughness" of surface, hydraulic radius, and hydraulic slope. It is purely empirical, and is based wholly upon experimental data obtained from open channels. The authors of the formula designed it for application only to open channels, and although it is now quite extensively applied to closed pipes, the data upon which this extension is based include few experiments upon large circular conduits. The authors have pointed out in the paper that no constant value of n (the coefficient of roughness) will bring their results, for either the steel or the wooden pipe, into even approximate conformity with the formula. The variation of n with the velocity is shown in the following table:

v.	n.	
	Steel pipe.	Wood pipe.
1	0.0135	0.015
2	0.0142	0.0137
3	0.0144	0.0131
4	0.0145	0.0128

The opinion was also expressed that the different values of n shown in Table No. 4 for new riveted pipes of various sizes could not be attributed to differences in roughness. Further, it was pointed out that the experiments of Mr. Adams at Astoria, on an 18-in. stave pipe, when compared with those made at Ogden by the authors on 72-in. pipe, show that the value of n for stave pipe must be regarded as varying with the diameter, if Kutter's formula is to be applied. The variation of n with the velocity has been noticed in various other cases, and is mentioned by several writers. In view of these facts, the opinion expressed in the paper, that Kutter's formula must be regarded as defective so far as applicability to large pipes is concerned, appears to be fully justified.

MEMOIRS OF DECEASED MEMBERS.

WILLIAM EZRA WORTHEN, Past-President and Hon. M. Am. Soc. C. E.*

DIED APRIL 2D, 1897.

William Ezra Worthen, Past-President of the American Society of Civil Engineers, died at his residence, in New York City, on April 2d, 1897, after an illness of some months.

Mr. Worthen was born at Amesbury, Mass., March 14th, 1819; his father, Ezra Worthen, having been one of the projectors of Lowell as a center of manufactures, and the first Superintendent of the Merrimac Mills from date of June, 1822. William E. Worthen, after having been fitted for college by I. B. Hayward, of Boston, was entered at Harvard, and graduated from that institution in 1838.

He commenced the profession of civil engineering under the tutorship of S. M. Felton, an assistant in the office of the then prominent engineer, Colonel Loami Baldwin; and the day after graduation from Harvard was put to work under the direction of George R. Baldwin, measuring the flow of the water used at the Merrimac Mills. With Mr. Baldwin he went to Boston and was employed in the surveys and brook measurements for an increased supply for the Jamaica Ponds Water-Works, a private enterprise for supplying Boston with water by gravity. He then returned to Lowell, and, under the late James B. Francis, Past-President and Hon. M. Am. Soc. C. E., was engaged for some time in general hydraulic and mill work.

In 1840 Mr. Worthen was with Mr. G. W. Whistler on the Albany and Stockbridge Railroad, commencing with the preliminary surveys of the road and remaining until its completion; 7 miles of the construction work having been under his immediate charge. Returning to Lowell he was again engaged with Mr. Francis in hydraulic work and the construction of the lower end of the Northern Canal; he then designed and built a dam and the mills on the Suncook, at Suncook, N. H. He also designed and constructed a dam and mills for the Boston Manufacturing Company, on the Charles River at Waltham, and the Suffolk, Tremont, Lawrence, Appleton and Hamilton Mills at Lowell. He reported on the water supply of Lowell, and for a time was in charge of the cotton mills and machine shops of the Boston Manufacturing Company, as Acting Superintendent.

* Memoir prepared by Julius W. Adams, George S. Greene, Henry Flad, Joseph P. Davis and Alphonse Fteley, Members, Am. Soc. C. E.

After a visit to Europe, in 1849, Mr. Worthen came to New York City and to some extent devoted himself to architectural work, building the structures at No. 200 Broadway. At the same time he took part in editing several mechanical publications. In 1851 he was in charge of the cotton mills and machine shops of the Matteawan Company at Fishkill; but, returning to New York in the following year, he was again engaged as an architect, designing and constructing, among other buildings, the bindery of the Appleton Publishing Company, in Franklin Street, and their present works in Williamsburg. Later, he was Engineer of the New York and New Haven Railroad and Vice-President of that road until 1854, under Mr. Robert L. Schuyler.

He designed and built the dam across the Bronx River at West Farms; and then, opening an office in New York City, supplemented his architectural and engineering practice by constructive iron-work and steam heating for buildings. He was engaged for a long time in general engineering work; building the dam across the Mohawk at Cohoes, N. Y., testing steam pumping machinery at many points, and designing pumping engines for Mr. Kirkwood for his reports on the water supply of Cincinnati, and of St. Louis. He devised floating grain docks for the same engineer at the Jersey City depots of the Erie Railroad Company, and was Sanitary Engineer of the Metropolitan Board of Health, of New York City, during its whole existence, 1866–1869.

Mr. Worthen had much practice in the measurement of the flow of water in canals, reporting upon this class of work for Paterson, Trenton, Passaic and Indianapolis, among the more important. He finished the Southern Boulevard in New York City; built the first pumping engine at High Bridge, New York City; the engine at Rockville Center and many pumping engines of smaller capacity; extended the water supply of the City of Cohoes and finished the Water-Works of Long Island City.

As Consulting Engineer he, at various times, examined and reported upon the Indianapolis and the Hartford Water-Works, the condition of the docks of New York City, the Riverside Park, the extension of the streets of Brooklyn, the selection of pumping engines for Boston, also the testing of large pumping engines at Brooklyn, Lawrence, Jersey City, Philadelphia, St. Louis, Milwaukee and other points. For this class of work his services were in very general demand for many years. In 1890 and 1891 he served as Chief Engineer of the Chicago Main Drainage Canal. He was Engineer of the later Commission on Rapid Transit in the annexed district of the City of New York, and reported upon the water-power of the Falls of St. Anthony, Minn.

He served on a committee of the Society appointed to investigate the failure of a dam at Mill River, Williamsburg, Mass., his associates

being James B. Francis and Theodore G. Ellis. Their report will be found in *Transactions*, Vol. III, page 118.

Mr. Worthen was a frequent contributor to the professional discussions of the Society, and presented the following papers: "Improvement of Sedimentary Rivers;" *Transactions*, Vol. XX, page 230; "Concrete Sewers at Mount Vernon, N. Y.," Vol. XXIV, page 393; "Steam Heating," Vol. XXIV, page 206.

This very incomplete sketch will give some idea of the varied practice which had occupied Mr. Worthen during a long and most active professional life, there being scarcely a branch of civil or mechanical engineering wherein his professional fitness has not been conspicuous in a marked degree. It was not his lot to project or carry to completion great works of internal improvement, such as challenge the admiration of the unthinking public, but in a very unobtrusive way he continually rendered that essential service toward the furthering of enterprise, without which the best conceived projects would prove abortive. To a remarkable power of rapid generalization, seemingly incompatable with painstaking accuracy, he united an almost intuitive perception of the requisite expedients of detail and design. His quickness in technical analysis, combined with the before-mentioned qualities, rendered possible the successful completion of many important works with which, owing to a forgetfulness of self, his name is scarcely associated. A retentive memory, to sift and treasure the facts in science and art which extended study had opened up to him, the tact of judicious selection and application, originality and boldness at times bordering on audacity, and a positiveness that silenced all opposition, have been the characteristic features of his long practice.

Whatever Mr. Worthen may have owed to study or experience in the arts of construction, his acknowledged genius (using the word in its strictest sense) stamped him as an engineer, *nascitur non fit*.

Mr. Worthen possessed an overflowing vein of wit and humor; which served to temper the asperities not unusual in professional debates; this, coupled with a kindliness of disposition which could see nothing in others, to speak of, but what was commendable, led to his friends being numbered only by his acquaintance. His social relations were of the happiest kind, and his memory will long be prized by all who knew him.

He married Miss Margaret Hobbs, of Boston, who survives him.

Mr. Worthen became a Member of the American Society of Civil Engineers December 4th, 1867. He was elected President of the Society January 19th, 1887, and served one term; he was made an Honorary Member of the Society April 4th, 1893.

BURR KELLOGG FIELD, M. Am. Soc. C. E.*

DIED JANUARY 13TH, 1898.

Burr Kellogg Field was born at Auburn, Ind., on the 5th day of May, 1856. Soon after his birth his parents moved to Malden-on-the Hudson, in New York State. He prepared for college at the St. John's Military School at Sing Sing, N. Y., and entered the Sheffield Scientific School of Yale University in the fall of 1874, graduating with the Class of 1877. His college career was not marked with any extraordinary incidents except his great popularity among his associates. Owing to his exceedingly affable, pleasant and generous ways he was christened by his associates "The Cherub," by which name he was known among his intimates until the last day of his life. Mr. Field's parents were wealthy, and he was denied nothing during his college course which would add to his happiness or contentment or in any way assist him in the prosecution of his studies. Ten days before commencement his father failed in business, and Mr. Field graduated from college in a worse than penniless condition, being obliged to borrow money from his associates to pay his ordinary student's bills in and about New Haven.

Burr K. Field commenced his engineering career after graduating from the course of civil engineering at Yale University as a water boy to a section gang on the Baltimore and Ohio Railroad at the salary of $3 per week. Every engineer knows of the struggle for existence of the young engineering graduates during the period of 1877, 1878 and 1879, and Mr. Field's case was no different, except that circumstances forced him to provide for the support of his aged father and mother and younger brother and sister out of his scant earnings.

In July, 1878, he entered the service of the St. Louis and San Francisco Railroad, in the department of tracks, bridges and buildings, where he remained until September 10th when he entered the employ of the Denver and Rio Grande Railroad as a rodman on the New Mexico and Colorado Division. After leaving the Denver and Rio Grande Railroad he was engaged as a rodman on the construction of the Omaha extension of the Old North Missouri Railroad until August, 1879, when he was appointed as a rodman on the Wichita extension of the St. Louis and San Francisco Railroad, in which place he remained until the spring of 1880, occupying successively the positions of rodman, leveler and topographer.

In the spring of 1880 J. F. Hinckley, M. Am. Soc. C. E., was detailed by the St. Louis and San Francisco Railway Company to

* Memoir prepared by Charles M. Jarvis, M. Am. Soc. C. E.

make surveys from what is now Monett, Mo., to Van Buren, Ark., and Mr. Field was engaged by him as topographer. He remained with Mr. Hinckley until the completion of the preliminary surveys. Much of the country traversed was wild and broken in the extreme, and little or nothing was known about it previous to these surveys. Mr. Field continued as topographer on location, under A. P. Man, M. Am. Soc. C. E., principal assistant engineer, and later under Mr. M. N. Randall, locating engineer. The work covered the territory between Fayetteville, Ark., and Van Buren, Ark., the line crossing the Boston Mountains, near where the present station of Winslow is located. Later Mr. Field was placed as engineer in charge of masonry, trestles and bridging on the line between Winslow and Van Buren. The masonry and bridging on this portion of the line is important in character, the streams crossed being rapid torrents, and the foundations being difficult to sink. Under his direction there were constructed three iron trestles ranging from 400 to 700 ft. in length and from 100 to 120 ft. in height, besides some twenty spans of Howe truss bridges resting on masonry piers. Upon the completion of this work in the summer of 1881, Mr. Field joined one of the surveying parties under the charge of R. L. Van Sant, M. Am. Soc. C. E., to complete the location between Van Buren and Fort Smith, and to make preliminary surveys through the country from Fort Smith to Paris, Tex. He remained in this position until February, 1882, when he was appointed Assistant Engineer on the Northern Pacific Railroad.

James Dun, M. Am. Soc. C. E., now chief engineer of the Atchison, Topeka and Santa Fé Railroad, who was at this time chief engineer of the St. Louis and San Francisco Railroad, speaks of Mr. Field in the following terms:

"My recollections of Mr. Field are of the most favorable nature, and I shall always retain the highest impressions of his character, professionally and personally. During the time he was engaged upon the St. Louis and San Francisco Railroad he was associated with men more than ordinarily bright, and who have since all won an honorable place in the profession which they adopted; and among them all I know of no one who promised to attain more than he. He was universally liked by his associates and by all who came in contact with him, and his untimely taking off was deeply regretted by all of his earlier friends and associates in the West."

From February, 1882, to November, 1883, Mr. Field was Assistant Engineer on the Northern Pacific Railroad, part of the time in charge of the construction of the Yellowstone Division, and later in charge of the tracks and bridges in the construction of the 52 miles of the National Park Branch, connecting the Yellowstone Park with the Northern Pacific.

On January 17th, 1884, he was appointed by Mr. John D. Estabrook, then Chief Highway Commissioner of the City of Philadelphia,

to the important position of Superintendent of Bridges in the Highway Department of the City of Philadelphia, "which position he filled in a thorough and conscientious manner, ever looking out for the public welfare regardless of party pressure or selfish ends."

Two years later in 1886 Mr. Field accepted an appointment as Assistant Engineer of the Berlin Iron Bridge Company, of East Berlin, Conn. His advancement with this company was very rapid, and at the time of his death he occupied the important position of Vice-President, having full charge of the making of all contracts. During Mr. Field's connection with this company its business was much extended, and its product introduced into all parts of the world. In achieving this Mr. Field had no small part, and his death has been a severe loss, not only to the company, but also to his associates.

Mr. Field was an indefatigable worker, not only for the corporations which he so faithfully served, but in every walk of life. He was a friend of his fellow-men, a devoted and earnest worker in the church, a staunch friend to the cause of temperance, and active and energetic in every public position that engaged his attention. To quote from the words of a friend:

"There is a vacant place—yes, not one but many vacant places in our midst, and his absence from these we know and feel is our great loss. There are vacant posts of duty which were always well filled. How many and varied they were! The posts of great business responsibility; of influence in Church work; of prominence in the efforts for good citizenship. The earnest and conscientious life which occupied these places has been taken from us, and we feel our loss. But not only in the large and more important positions do we know that a loss has come, but such was the largeness of his life and efforts that within the small places there is a vacancy; there are vacant places in those higher realms of life's experience—friendship has lost a true friend; earnest and conscientious service in the Church has lost a bright example; Christianity has lost a good Christian. Think of what is good and bright—he stood for that. Think of what is true and earnest—he stood for that. Think of what is unselfish and noble—he stood for that. Think of what is tender and sympathetic—he exemplified that. From whatever standpoint we view his life, as a business man, a citizen, a member of the Church, a friend, or the head of a family, we behold one who stood for what was best and true in all these. To say such of Burr K. Field is not to speak flattery or extravagant eulogy, but only to speak the sober truth."

In the flush of manhood he was taken away without an instant's warning.

While engaged with the St. Louis and San Francisco Railroad, he became acquainted with Miss Juanite Bourland, the daughter of Doctor Bourland, of Van Buren, Ark., to whom he was wedded on the 5th day of May, 1886, and who, with two children, survives him.

Mr. Field was elected a Member of the American Society of Civil Engineers on October 1st, 1884.

THOMAS DAVIS LOVETT, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1897.

Thomas Davis Lovett, son of Captain Pyam Lovett and Lucy Davis Lovett, was born June 10th, 1823, in Beverly, Mass., where he grew to young manhood, receiving his education from the Beverly Academy, and later in New York City. He was descended, both on the paternal and maternal side, from honorable ancestors, and was a great grandson of Benjamin Lovett, one of the one hundred and five minute men who marched from Beverly to Concord, October 19th, 1775. While at the academy his father had him employ the usual Wednesday and Saturday half holidays in learning the routine of book-keeping and commercial transactions as they occurred in the Beverly Bank, of which Captain Lovett had for many years been president. It was the father's expectation that the son would succeed him in that office of trust and responsibility, but after his return from school at New York he began the practical study of civil engineering under Frank Edwards, at Beverly, about 1841 or 1842.

With that principal, his first railroad employment was in the construction of what is now the Gloucester Branch of the Boston and Maine Railroad, and next in the construction of the road then known as the Portland, Saco and Portsmouth Railroad, now also a part of the Boston and Maine system. Soon afterward he was employed on the Troy and Boston Railroad, having, for a time, at least, responsible charge of maintenance of certain portions constructed, this leading at a later period to his rebuilding the bridge masonry on the branch to historical Bennington.

The surveys for the eastern portion of the Hoosac Tunnel Line, known as the Troy and Greenfield Railroad, incorporated under the auspices of the Commonwealth of Massachusetts, followed in 1848 and 1849, or later. A part, at least, of these surveys, and possibly those for the entire line, were made by Mr. Lovett. He and a young English draughtsman, a Mr. Dillon, prepared extensive and elaborate maps of the line, including the great tunnel, which were, unfortunately, all destroyed by fire some years after. While this and the corresponding work was under way, he moved from Greenfield, Mass., to Shelburne Falls, thence to North Adams, and thence, upon the completion of these undertakings, to Troy, N. Y., where he built the first street-car line of that city.

In 1853, Mr. Lovett moved to Chillicothe, Ohio, with Captain Archibald Kennedy (formerly of the Vermont Central Railroad), and,

* Memoir prepared by W. B. Ruggles and Samuel Whinery, Members, Am. Soc. C. E.

associated with John Waddle, M. J. Becker and Jacob Blickensderfer, all of whom, in later years, were prominent in railroad and State improvements, was soon employed in the location and construction of the Marietta and Cincinnati Railroad. Mr. Lovett, as assistant engineer, had charge of part of the construction from Chillicothe, west, toward Blanchester, and of surveys from Blanchester to Cincinnati. The lines of these surveys were direct and permitted of lighter grades than those finally adopted; but, as much of the projected location was in close proximity to the Little Miami Railroad, that road was for many years used as an entrance into the city, and the final location to Cincinnati, finished in 1869, was in a great measure determined by the necessity of reaching connecting lines in Cincinnati.

While making these surveys he lived at Goshen, Ohio, but had an office on Third street, Cincinnati, not far from that occupied by him during the last years of his life, nearly forty-five years later.

In 1855, upon the completion of his work on the Marietta and Cincinnati Railroad, Mr. Lovett went to McArthur, Ohio, on the line of that road, and associated himself with others for the purpose of developing the coal fields of the vicinity, and, incidentally, he laid out and built up the town of Zaleski. Later, he removed to Chillicothe, but continued his connection with the mines until they were sold to English parties. While these enterprises did not prove financially very profitable to Mr. Lovett, he there acquired the experience and practical knowledge of coal operations that were subsequently valuable on the Ohio and Mississippi Railway and the Cincinnati Southern Railway.

Immediately after the sale of these mines, and in February, 1860, he removed to Cincinnati, were he was for a brief time employed in the City Water-Works Department. His early experience on the city passenger tramways of Troy was soon the means of his engagement for the construction of the Third Street line in Cincinnati, the first street railroad built in that city, and, shortly afterward, of the John Street line, the next road completed.

It was at a time when the popular prejudices of the city were against these, so far locally untried, public conveniences, and it required, not only his ability as a constructing engineer, but much diplomatic and skilful management to carry the project to successful completion.

Following these experiences, and in the same year (1860), Mr. Lovett entered the service of the Eastern Division of the Ohio and Mississippi Railroad Company as Chief Engineer, and continued with that road, with one interval of other service, until 1872. At the time of his appointment, George B. McClellan, the famous general of a few years later, was General Superintendent of the road, and placed implicit confidence in Mr. Lovett, giving him complete charge of the

engineering of the road, to the extent of not even giving any direct or specific instructions in so grave an emergency as the collapse of a bridge.

Mr. Wm. F. Biddle, who was at that time Division Superintendent, has, in a late memorandum, recorded the thoroughness with which Mr. Lovett looked after the details of his work, and says: "I have never forgotten the appreciation and admiration that were inspired by Lovett's ability, capacity and devotion to duty during that very trying time."

During Mr. Lovett's first term of service with the road many improvements were made, and many more wisely projected, but the emergencies of the war and the delay incident to the liquidation and reorganization of the company were such that the important measures urged by Mr. Lovett could not be carried out, but time has developed their wisdom.

Mr. Lovett left the service of the road, and, in 1864, went to Boston, and there acted as engineer for the Messrs. Stanton, who were engaged in general contracting work.

He maintained, however, an active interest in the affairs of the Eastern and Western Divisions of the Ohio and Mississippi Railroad Companies, which were, in 1867, reorganized as one corporation. The immediate occasion of his return to the West was the failure of the bridge over the Great Miami River. Mr. W. D. Griswold was, at the time, president, and he at once telegraphed Mr. Lovett to return; this he did on November 1st, 1867, and for seven months gave unremitting attention to the reconstruction of the masonry and superstructure of this bridge.

Then followed a period of active reconstruction and improvement on the road, in the course of which all the principal bridges were renewed in iron, the heaviest grades reduced, coal fields developed, and the terminal facilities of East St. Louis and Cincinnati enlarged, making such provision for the future business of the road that no radical changes and but few extensions or rearrangements have since been found necessary.

The branch line from North Vernon to Louisville was built under Mr. Lovett's direction and was completed in 1871. This is a monument that, even without his later and more extensive undertakings, would have established his reputation for judgment and ability; for it is seldom, in a region where 1% grades are considered very favorable, that a cross-country direct line 50 odd miles long can be so located as to require only 0.8% grades, and these falling, with two short exceptions, in one direction, making practically a continuous grade from the high ground at North Vernon to the Ohio River.

There still remained to be made one important change to put the road on a basis which would maintain its supremacy. The Ohio and

Mississippi Railroad, constructed by engineers and contractors coming from the Erie Railway, was laid with the corresponding 6-ft. gauge, and formed, with that road and the Atlantic and Great Western, a continuous broad-gauge line from New York to St. Louis. It was Mr. Lovett's final duty to change to standard gauge of 4 ft. 9 ins. the western section of these systems.

The Ohio and Mississippi was the first railroad of any considerable length to attempt the change of gauge of the whole line without any serious interruption of the train service, and Mr. Lovett was a pioneer in this as in many of his other undertakings. The change from 6 ft. to 4 ft. 9-in. gauge was so considerable that it was necessary to move both rails.

The most complete and careful preparations were made, and all the minute details were worked out and provided for by Mr. Lovett. There still exist among the papers of the company the full and complete printed instructions and tables prepared by him for the work. On Sunday, July 16th, 1871, the change of gauge was made on the Louisville branch and furnished a preliminary test of the plans adopted. One week later, on Sunday, July 23d, 1871, the gauge of the main line was changed from Cincinnati to St. Louis, a distance of 335 miles, in the remarkably short time of seven hours. It was at the time justly considered a remarkable feat, and its successful accomplishment illustrated Mr. Lovett's great executive ability. The experience gained in this work rendered Mr. Lovett's assistance valuable to other roads whose gauge was changed later, and he was called in consultation in a number of cases.

Late in 1872 his connection with the Ohio and Mississippi Railway Company ended, and in October of that year he entered the service of the City of Cincinnati as Principal and Consulting Engineer for the construction of the Cincinnati Southern Railway.

Surveys for this long-contemplated thoroughfare to the South had been made under the direction of W. A. Gunn, and the necessary State and municipal legislation secured and appropriations provided. Of these experimental lines, some were to Nashville and some to Knoxville and many to Chattanooga, and the first matter to be determined, after Mr. Lovett took hold of the project, was the selection of a line for final location. Each of the routes had its earnest advocates, the choice lying principally between that to Knoxville and those to Chattanooga. This problem was solved, by the adoption of one of the lines to Chattanooga, and the work of construction was soon under way, the first contract for grading having been signed in December, 1873. The magnitude of the undertaking is indicated less by the total length of the new line, 336 miles, than by the character of that line. To provide adequately for the construction of nearly 5 miles of tunnels at twenty-seven points, fifty-seven bridges, aggregating more than 2 miles in

length and 3 miles of viaducts, might, in a large part, be delegated to subordinates, as becomes the discreet and judicious manager of large affairs in these busy times; but Mr. Lovett, in addition to the usual general supervision, looked closely into the details of all this work. Not only was the construction of the Cincinnati Southern Railroad one of the largest and most important public works of the period, but it must be remembered that some of the structures were at the time unique in engineering experience and practice.

High Bridge, over the Kentucky River, for instance, was then not only the highest bridge on the continent, and the highest railroad bridge in the world, but it was probably also the first bridge built on the cantilever principle without falsework. While the direct credit for its design and construction belongs to the late C. Shaler Smith, the opportunity might never have been afforded to him had not the chief engineer of the road been a man of broad views and bold engineering conceptions.

The channel span of the bridge over the Ohio River at Cincinnati was, at the time it was built, decidedly the longest trussed girder span in the world, and exceeded what at that time was considered the extreme practicable limit of iron truss spans. Its history furnishes another illustration of Mr. Lovett's professional courage.

The Ohio being a navigable stream, the location for the proposed bridge was referred to the usual Board of United States Engineers. The law required an opening of 400 ft., but a controversy arising as to whether the crossing should be at Smith Street, as selected by the Board, or at Ludlow, as desired by the representatives of the river interests, the Board gave, as an ultimatum, the alternative of the required 400-ft. opening at Smith Street or of a 500-ft. opening at Ludlow, stating in effect at the same time that they considered a span of that length impracticable. Mr. Lovett, however, at once chose the Ludlow site, and the span was made 515 ft.

Considering that in addition to heavy and costly work on other portions of the line, that part of the road from the vicinity of the Cumberland River to the Emory River, crossing the Cumberland Plateau, is the heaviest and most expensive continuous stretch of railroad of equal length in America, some idea may be formed of the magnitude of the enterprise and the difficulties that had to be successfully met.

In December, 1876, when nearly the whole of the work had been let and the greater part of it completed, Mr. Lovett tendered his resignation as principal and consulting engineer, and soon after went abroad, residing for some time in Paris where there was at the time of the Exposition of 1878 an Exhibit of the Cincinnati Southern Railroad. This display led to his being engaged by French capitalists to make surveys for a line of railroad from Jerusalem to Joppa. It is to be regretted that little is known of his experiences there, further than that

he endeavored to have all points of historical interest, as far as possible, rendered accessible; but, that a practicable line was developed, is shown by its later construction.

Mr. Lovett, however, found the work very trying, physically, and one effect, little suspected by even those who knew him well, was the loss of the use of one eye. Sick with a prevailing fever, he returned to the United States, and soon after opened, on Third Street, in Cincinnati, an office as Consulting Engineer.

While connected with the Cincinnati Southern Railway Mr. Lovett had become interested in the possibilities of the coal fields along the road, and, shortly after his return, invested in the "Beaver Creek and Cumberland River Coal and Railroad Company," which investments he held until his death. He was actively engaged during the succeeding years in the management and development of this property, and of his real estate at Beverly, Mass., and in Kansas.

In the intervals of professional engagement, he, in one way or another, had been connected with many other enterprises. While acting as Chief Engineer of the Ohio and Mississippi Railway, he was made engineer for the Front Street Connecting Railway in Cincinnati, and in that capacity designed the combination rail used in the track. He was Consulting Engineer for the first of the many inclines which were a distinguishing feature of Cincinnati suburban development. He had a remote connection with the erection of the substructure of the Sabula Bridge of the Chicago, Milwaukee and St. Paul Railroad over the Mississippi River, and an active interest in the erection of the substructure for the St. Charles Bridge over the Missouri River. He traveled much, both professionally and for recreation, and in the course of a busy life had been in every quarter of the globe. The attraction of life increased with his years, and to the last he participated in the many gatherings of professional men, where he was always a welcome guest.

Mr. Lovett selected his subordinates with rare judgment. On the Ohio and Mississippi and the Cincinnati Southern Railways he had, as his assistants, T. M. R. Talcott, R. L. Read, Isaac Smith, R. L. Engle, G. Bouscaren, Samuel Whinery, H. R. Weeks, George B. Nicholson, W. P. Clark, Cabell Breckinridge, Robt. B. Stanton and Charles L. Strobel, all of whom have since been prominent as engineers and nearly all of them members of the American Society of Civil Engineers, and active in its affairs. It is doubtful if a better corps of engineers than that on the Cincinnati Southern Railway was ever organized.

Mr. Lovett was married September 8th, 1846, to Sarah Frances Folger, of Nantucket, who survives him. Of four children, two are living, Mrs. G. W. Pohlman and Mr. Pyam Lovett. He was a man of strong and positive character, great energy and of superior executive

ability; quick to decide and forceful to execute; bold and courageous in his opinions and frank and plain spoken in expressing them; fair and honorable in his business relations and loyal to his friends. Intolerant of deceit and trifling, he was a just and considerate superior, reposing confidence in his subordinates, but holding them to a strict accountability in their professional work, and always ready to appreciate and give full credit for faithful service and meritorious work.

Mr. Lovett was elected a Member of the American Society of Civil Engineers on May 3d, 1871.

MIRTILIANO SICARD, M. Am. Soc. C. E.*

DIED MARCH 17TH, 1896.

Mirtiliano Sicard was born at Bogotá, Colombia, on July 2d, 1840. He was the son of Pedro Sicard Ponz, a Frenchman, and Maria Joséfa Pérez Bonalde, a Venezuelan.

He studied literature and philosophy at the Jesuits' College at Bogotá until 1854, and then attended the Military School in the same city, where he studied engineering.

In 1860, with the co-operation of his brothers, he founded a college at Lérida, which college was afterward successfully transferred by them to Ibagué.

In order to devote his time to commercial affairs, he some time later severed his connection with the college. Soon afterward he married Enriqueta Urdaneta.

His career as an engineer began at that time, since, in consequence of various contracts with the government, he brought from the United States iron bridges of various types and erected them over the rivers Combeima, China, Fotare, Coello, etc., works which honor the government which defrayed the cost. These works are of the first rank, and are a credit to the engineer who erected them. The bridge erected by Mr. Sicard over the Coello at the pass "El Chicoral" ranks with the best found in the Department. This has been acknowledged by competent persons, and especially by Joaquin Esguerra in his geographical dictionary, in which special mention is made of the bridges erected by Mr. Sicard.

* Memoir prepared by Julio Sicard, Esq.

Mr. Sicard was intimately connected with the improvements which were undertaken in the Department under various governments, and many of the public works were carried out under his management.

Returning from a recent trip to the United States, he brought back to Colombia the machinery for a mineral assaying office, which was established at Ibagué. Soon after, he had two iron boats constructed in the same country, which were intended for the rivers Saldaña and Magdalena. He also had several bridges constructed.

At all times Mr. Sicard was a useful member of society. He was constantly employed, and the offices of Secretary of the Treasury, Rector and Trustee of the Colegio de San Simon de Ibagué and Secretary of Public Instruction in the Department of the Tolima were successively filled by him. In this last employment he was overtaken by death, at Girardot, on the 17th of March, 1896, at a time when, by order of the Government, he was starting on the inspection of the work on the Railway of the Tolima.

Mr. Sicard was elected a Member of the American Society of Civil Engineers on January 2d, 1882.

HENRY CLAY YEATMAN, Jun. Am. Soc. C. E.*

DIED DECEMBER 20TH, 1896.

Henry Clay Yeatman, the eldest son of Colonel Henry Clay and Mary Brown Yeatman, was born in the City of Nashville, Tenn., March 2d, 1866. On both sides (mother and father) he came from that old original sturdy stock that made Tennessee famous in war and renowned in the councils of the Nation, and whose word was their bond. Colonel Yeatman was on the staff of General Leonidas Polk (the Fighting Bishop) during the late rebellion and is still living in Maury County, Tenn. Mrs. Yeatman (Mary Brown Polk) was a daughter of General Lucius Junius Polk, one of the first settlers of Maury County, Tenn., and she was also a great-niece of President James Knox Polk.

Henry Clay Yeatman, Jr., passed his early childhood at his home in Maury County, Tenn., and attended the country schools in the neighborhood until he was fourteen years old. He then went to school at Winchester, Tenn., for one year and for a time attended the University of Tennessee at Knoxville.

Leaving the University of Tennessee he went to Texas, and from April, 1883, until the following September was Rodman on the Galveston, Harrisburgh and San Antonio Railway on the location of several branches. He then attended Lehigh University, Pennsylvania, until January, 1886, and, returning to Texas, secured a position as Levelman on the Mexican International Railway in Mexico, and held this position until May, 1886. From May, 1886, until January, 1888, he was Assistant Engineer in charge of bridging and track-laying on the same road.

From 1888 to 1889 he was Civil Engineer in charge of the Sabinas coal mines in Mexico. From May, 1889, until August of the same year he was Engineer in charge of location and construction of irrigating works for the Eagle Pass (Texas) Irrigation and Water-Works Company. From August, 1889, to June, 1892, he was Assistant Engineer on the location of the Mexican Pacific Railway in the States of Tabasco and Chiapas, Mexico.

His health failing at this time, in the low, hot States of Tabasco and Chiapas, he returned for a time to the United States. In 1893 he went to work again on location on the Mexican International Railway, remaining until the fall of 1894, when he went to Guatemala, working for the contractors on a new railway project.

While in Guatemala he was directed to inspect certain contracts for piling, some 60 miles down the coast. He set out in an open boat with one companion, expecting to keep close to the shore, but a

* Memoir prepared by Samuel Killebrew, M. Am. Soc. C. E.

gale striking the frail craft, carried it far from land, capsizing and sinking it. Catching a loose yawl that was on the boat they passed four days and nights in a drenching rain, all the while bailing the water out of the yawl, and were without food or drink until they were rescued by a passing Norwegian bark, which landed them at Pensacola, Fla., eight days later. This trip nearly cost Mr. Yeatman his life, but being of strong frame and robust constitution he soon rallied and returned to work on location on the Mexican International Railway, and there continued until September, 1896.

He was absolutely fearless, never hesitating in any emergency, regardless of the danger, but on the other hand, he seemed at all times to court danger. He was generous to a fault, dividing his last cent with friend or foe.

He always took a great interest in all that were under him and never grew tired in trying to advance them in every way possible. He was happiest when crowded to full capacity with work and was a most agreeable assistant. He spent the last few years of his life in Brownsville, Tenn., where he made a host of friends who mourned his sudden and unexpected death.

He died from a hemorrhage of the lungs, December 20th, 1896, at the Peabody Hotel, Memphis, Tenn., where he had gone to spend a few days.

Mr. Yeatman was elected a Junior of the American Society of Civil Engineers on May 7th, 1890.